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Assessing and Enhancing the Durability/Longevity Performances of Highway Bridges

HRC Research Project
2-13506
Final Report

Prepared by

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ABSTRACT

This is the final report on Highway Research Center Project 2-13506. The long range objective of this investigation was to achieve enhanced durability/longevity performances of future new and rehabilitated highway bridges. Short term objectives to achieve this end were: (1) to assess the durability/longevity performance of past highway bridges in Alabama, the U.S., and other countries in order to identify "how/why" these bridges "failed," (2) to investigate the major parameters influencing bridge durability/longevity in order to assess their importance in achieving bridge durability/longevity in Alabama, and (3) to identify actions which should be taken during the planning, design, construction, and maintenance phases of highway bridges to enhance their durability/longevity.

Assessments of past bridge durability performances were made based on data available in the literature, data available in the historical bridge records of the Alabama Department of Transportation (ALDOT), results of a mail survey/questionnaire, and results from personal interviews conducted. The investigation was limited to steel, reinforced concrete and prestress concrete bridges of structural forms that are in wide use, and, to the extent possible, those that will probably be widely used in the future.

The primary factors causing deterioration and hampering durability/longevity performances of highway bridges in Alabama are identified in the report, and actions to mitigate these factors are recommended. The factors and actions to be taken are presented in the separate tables for each of the major participating bureaus of the ALDOT, i.e., the Planning and Design Bureau, the Bridge Design Bureau, the Construction Bureau, and the Maintenance Bureau.

ACKNOWLEDGEMENTS

The financial support of this work by the Highway Research Center at Auburn University is gratefully acknowledged. In-kind support of the project by many persons of the Alabama Department of Transportation (ALDOT) at their central office in Montgomery, Alabama, as well as all nine Division Offices, is acknowledged and very much appreciated. Particular thanks are due Mr. Fred Conway, Bridge Bureau Chief, ALDOT, and Mr. Terry McDuffie, Bridge Maintenance Engineer, ALDOT for their continued assistance throughout the course of the project.

CONTENTS

1. INTRODUCTION

1.1 Statement of Problem

A large number of highway bridges in Alabama and in the nation are in need of major rehabilitation or replacement. Most of these bridges have performed adequately, but not in an outstanding manner in the area of durability. The Alabama Department of Transportation (ALDOT) would like to see replacement bridges be more durable and provide an improved life-cycle-cost. To accomplish this goal, it would be helpful to know "how/why" the old bridges are "failing." This would allow the identification of problem areas and areas in which current procedures and methods are working well. In turn, this would provide some direction for planning, design, construction, and maintenance improvements to enhance the longevity and durability of existing and future bridges.

With the need to replace many bridges on the horizon, along with the availability of much historical data on bridge durability performances (in Alabama, other states, and other countries), the time is ripe to stop and examine what is being done correctly or incorrectly in bridge planning, design, construction, and maintenance. This was the impetus for this investigation.

1.2 Study Objectives

The long range objective of this investigation was to achieve enhanced durability/ longevity in future new and rehabilitated highway bridges. The specific short term objectives of the investigation were as follows:

1. To assess the durability/longevity performance of past highway bridges in Alabama, the U.S., and other countries in order to identify "how/why" these bridges "failed."
2. To investigate the major parameters influencing highway bridge durability/longevity in order to assess their relative importance in achieving bridge robustness, and to estimate their variability and controllability in the planning, design, construction, and maintenance processes.
3. To identify actions which should be taken during the planning, design, construction, and maintenance phases of highway bridges to enhance their durability/longevity performances.

1.3 Plan and Scope of Study

Assessments of past bridge durability performances were made based on data available in the open literature (Chapter 2) along with data available in the historical bridge records of the ALDOT (Chapter 5). Only a subset of the ALDOT data was utilized in the assessment. Also a mail survey and personal interviews were conducted to assess the durability/longevity performances of ALDOT bridges (Chapters 4 and 6). Data from these sources were studied to assess durability performances of major bridge components and subcomponents to identify "how/why" highway bridges are failing.

The actions needed (Chapter 7) developed in the study are based on findings in assessing bridge durability performances, and recommendations of various researchers and agencies as found in the literature, from the mail survey, from the study of bridge historical records, and from the personal interviews conducted. The emphasis in these actions is on structural engineering details and overall guidelines for bridge planning, design, construction, and maintenance.

The investigation was limited to steel, reinforced concrete and prestress concrete bridges of structural forms that are in wide use, and to the extent possible, those that will probably be widely used in the future. Timber bridges and truss, arch, cable and other somewhat unique bridge structural forms and culverts were not investigated in this study. Bridge materials, while being extremely important to bridge durability, were viewed as a subject for a separate study and report, and detailed guidelines on material selection and use are not included in this report.

2. LITERATURE REVIEW

2.1 General Background

There are an estimated 577,710 bridges in the United States, and approximately 42 percent are deficient. Of these, 24 percent are structurally deficient and 18 percent are functionally obsolete [29]. Over 800 new bridges (built from 1985-1989) are already classified as structurally deficient. Unfortunately, it appears that most of these bridges are in the southeastern part of the U.S. as indicated in Fig 2.1. The FHWA states that the United States replaces their bridges on average at age 70 years [29].

The percent of highway bridge types - identified by their superstructure structural system materials - built annually has changed considerably between 1950 and the present time [24]. Prior to 1950 there were no prestressed concrete bridges in this country, and today they capture over 50 percent of the highway bridge market and the rapid rate of growth is continuing. This rather dramatic change in bridge superstructure material is graphically illustrated in Fig 2.2.

2.2 Bridge Durability Philosophy

A design consideration that probably receives too little consideration in bridge design, and in bridge concrete mixture design as well, is durability. In highway maintenance activities, bridge durability and longevity is of paramount importance. However, in the processes preceding maintenance activities, namely bridge planning, structural design, material selection/design and construction, bridge durability usually does not receive the

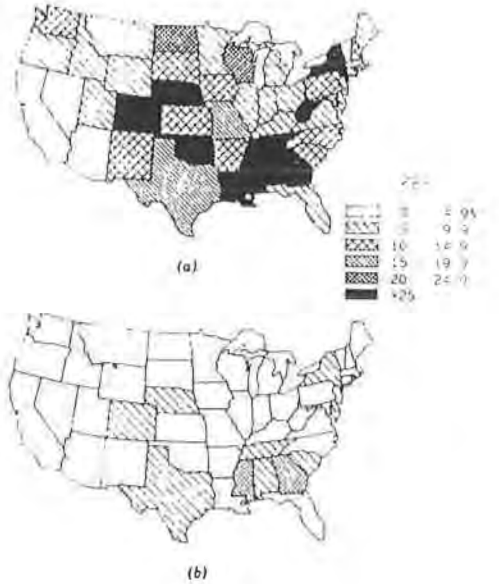


Figure 2.1 Percentages of Bridges Classified Structurally Deficient: (a) Built 1950-1987; and (b) Built 1980-1987 [23].

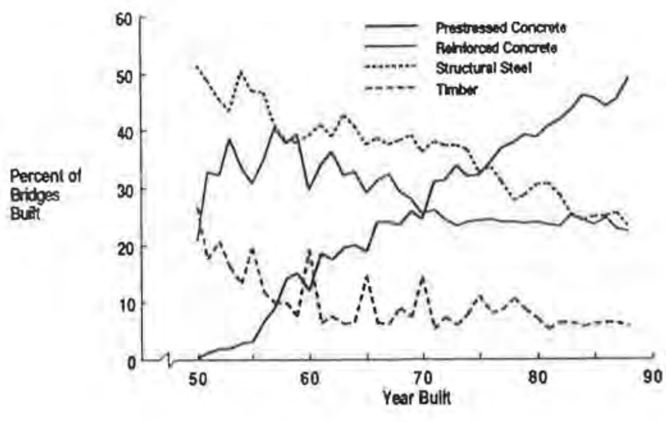


Figure 2.2 Percent of Bridge Types Versus Year Built as of May 1989 NBI [23].

attention that it should. Before discussing this further, it would be helpful to define what is meant by durability of highway bridges.

> Durability of highway bridges is considered to be the ability of the material/bridge to resist the forces of disintegration due to cyclic temperature and moisture changes, freezing and thawing, chemical attack, traffic loadings and other conditions that will comprise the bridge's natural and manmade environment for 100 years after construction.

Over the years, even though durability has not received the attention that it should, bridge durability in this country has not been bad. However, it has not been excellent either. Because of the large number of bridges that currently need major rehabilitation or replacement, the large dollar investment in our national infrastructure that this represents, and the fact that durability of our current highway bridges leaves something to be desired, it behooves us to take a close look at what we have been and/or are currently doing in the areas of bridge planning, design, construction and maintenance. This will allow us to identify needed changes in each area in order to produce bridges in the future with greater durability/longevity.

When one considers the consequences of a non-durable bridge, i.e.,

- Reduced service life and thus reduced life cycle cost
- Increased maintenance cost, rehabilitation cost, replacement cost to the owner in dollars and time and effort expenditures
- Reduced safety to the user
- Increased inconvenience and time and dollar cost to user
- Bad PR and reduced confidence factor of user for owner

it would appear that all significant bridges on major highway or roadway systems be designed for at least 100 year lives. Even in those cases where the highway or bridge may be abandoned or rebuilt because of other conditions before 100 years, it makes sense to spend the small additional dollars required up front to achieve a more durable and more maintenance free structure.

The time to begin thinking about durability/longevity is at the very beginning in the bridge planning stage. In general, the quality and future performance of a bridge are more strongly influenced during the planning and design phase of its life than at any other time. Figure 2.3 graphically illustrates this point. It should be noted that Fig. 2.3 is based on all CE oriented projects and is probably "flatter" in the operation period than would be the case for highway bridges. Nonetheless, as indicated in the figure, the planning/design phase is where we as engineers can have the greatest influence on bridge longevity.

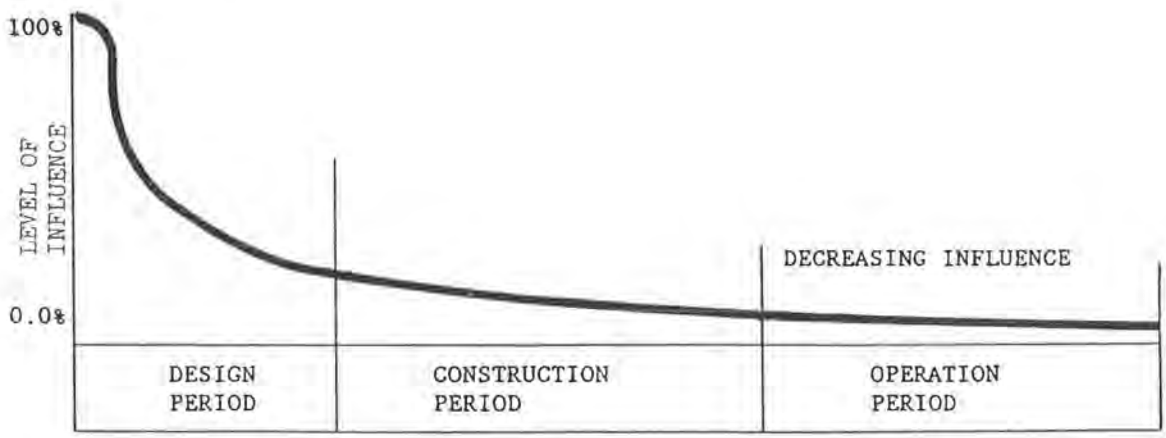


Figure 2.3 Opportunity to Influence Project Characteristics [8].

Once a bridge is designed and constructed, much of its eventually durability and longevity has been determined. However, during its operational life, there still remains numerous activities and decisions which have a significant impact on the bridge's durability and longevity. These activities and decisions relate primarily to the bridge's inspectability, maintainability, and repairability/ rehabability. Good periodic inspection is important to achieving longevity in bridges so that any distresses can be located and addressed early (however, an ounce of prevention is worth a pound of cure, i.e., design and construction durability/longevity into the bridge before it is placed into operation). In the area of inspectability, steel bridges are the easiest to inspect to determine incidences of distress or the bridges' structural state. It is much more difficult to see inside of concrete members to determine their state of soundness. However, advancements in technology are being made in this area. In the area of maintainability, both steel and concrete bridge structural members require periodic maintenance, i.e., painting for steel members, and waterproofing/sealing for concrete members. Of course, bridge joints, bearings, etc. require periodic maintenance for all bridge types. In the area of repairability/rehabability, steel bridges are the easiest to repair and are probably the easiest to rehabilitate and/or upgrade in capacity and/or width. If the rehabilitation work is that of replacing the deck (which is probably the most common type of rehabilitation), then it is easier to replace a concrete deck on steel girders than on concrete girders. In addition to being more difficult to detect deteriorations and to make repairs, quality in construction has a greater potential for variability for concrete bridges, and quality of the constructed bridge is closely related to durability. Hence, for a concrete bridge, it is of paramount importance that durability/longevity be designed and constructed into the bridge at the very outset.

2.3 General Mechanisms of Bridge Failure

To understand how to design for durability, one needs to understand how bridges fail. Although specific failure details and sequences are not agreed on, engineers usually agree on the general mechanisms of bridge failure. The description of the general mechanisms of failure presented in the following paragraphs is an abridged version of that given in Reference

26.

In physical terms, bridges deteriorate because of weather and traffic. Water corrodes steel and can scour away bridge foundations. Meanwhile, every car and truck that passes over a bridge causes it to flex. Excessive loads can cause cracks that destroy the structure's integrity.

Once a bridge has begun to deteriorate, the process of decay accelerates. The portions of metal beams that are under the most stress corrode more rapidly, and stress concentrations increase as the thickness of sound metal decreases. Similarly, damaged structural members have reduced load-bearing capacity and are more vulnerable to the effects of heavy traffic.

Simple problems, if allowed to progress unchecked, can lead to severe damage. Debris on a bridge deck, for example, may block drains, causing water to accumulate, or in cold regions, water may freeze inside the deck, cracking it. Deicing compounds form salt solutions that rapidly corrode reinforcing bars and other structural members.

Bridge drainage poses its own problems. Engineers have learned at great expense to direct water away from the structural elements and bearing surfaces, where the combination of salt and stress can destabilize a bridge in a few years. Yet complex drainage systems are expensive and require periodic maintenance to keep them from clogging.

Bridge substructures face problems similar to those of the superstructure and deck. In addition, they are often much more difficult to inspect and to repair. The Schoharie Creek Bridge in upstate New York, for example, collapsed in 1987 because flowing water had scoured away its foundation.

The primary locations and causes of bridge failures are graphically illustrated in Figure 2.4.

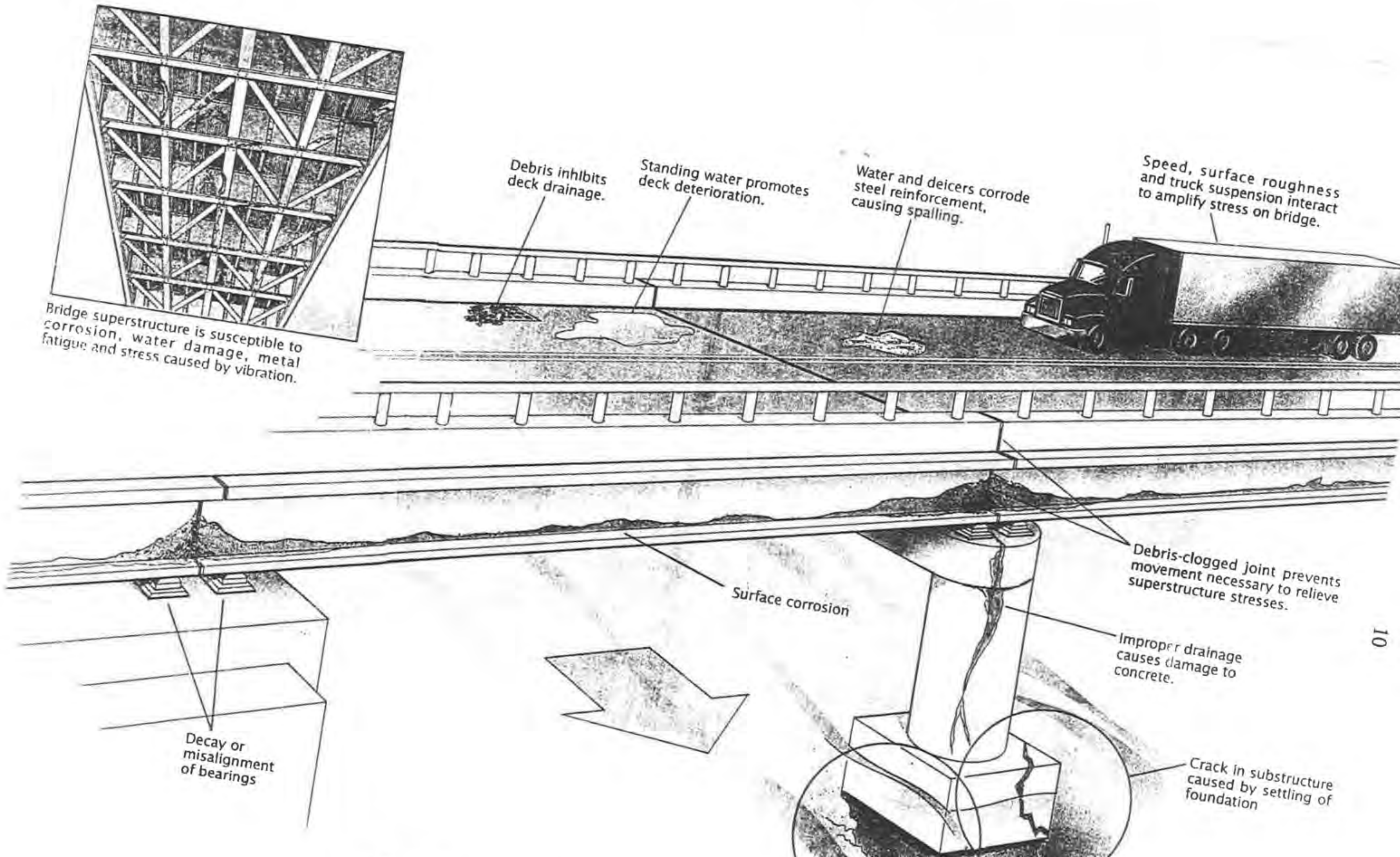


Figure 2.4 Graphical Illustration of How Bridges Fail [26].

2.4 Bridge Deck Deteriorations and Durability

Of all the components in highway bridge systems worldwide, bridge decks should probably receive the greatest special consideration and attention during the planning, design/material selection, construction and maintenance phases. The bridge deck environment is one of the worst possible exposure conditions for concrete. The decks are:

- Horizontal surfaces (which are less durable than vertical surfaces)
- Subject to alternate wetting and drying
- Subject to alternate freezing and thawing
- Subject to large solar thermal loadings with large thermal gradients
- Subject to increasingly large truck tire pressures
- Subject to highly concentrated studded tire loads in some locations
- Subject to applications of deicing salt and grit (fills expansion joints) in some locations
- Subject to impact and fatigue loadings from highway traffic
- Subject to rolling frictional loads and the associated wear and polishing actions

Additionally, bridge decks are continuous across the width of the bridge and may be continuous along the axis of the bridge as well. This leads to increased incidences of bridge cracking and in turn to decreased durability. Also many bridges are skewed or have a changing profile and these tend to decrease durability.

Four types of bridge deck deterioration are commonly cited,

- Cracking
- Scaling
- Wear/polishing
- Spalling

Each of the these is discussed in the following paragraphs.

Cracking. Some cracking is to be expected with concrete and is characteristic of the material because of its low tensile strength, its shrinking as it solidifies and hardens, and the volume changes that it exhibits in response to changes in humidity and temperature. However, the extent and severity of the cracking must be controlled by good structural design, material mix design and construction practices. Cracks provide a convenient means for water, deicing salts, fine particles and other undesirables to gain entry to the concrete deck. The fine particles can build up and result in accumulating damage during daily and seasonal temperature changes in the same manner as stopped-up expansion joints. Water entry into the concrete can lead to accelerated deterioration due to freeze-thaw actions. And, the entry of water and deicing salts usually cause some degree of rebar corrosion. The extent of rebar corrosion is primarily determined by the depth and quality of the concrete cover.

Cracks that follow the line of the rebar are much more potentially damaging (relative to cracks transverse to the rebar), because they provide convenient water and/or deicing salts access to the rebar for the length of the crack, and these cracks reduce the resistance of the concrete to spalling.

Scaling. Scaling is the flaking of surface motar and is often accompanied by loosening of surface aggregates. It is the result of frost deterioration of the surface concrete. Unfortunately, because of concrete consolidation efforts during pouring, concrete bleeding, and surface finishing work, the concrete at the surface of a bridge deck is normally the worst quality concrete in the deck. In turn the potential for deck scaling is primarily affected by the quality of the concrete at the surface of the deck.

Wear/Polishing. On urban freeways, wear in the wheel tracks is a source of bridge deck deterioration and can require premature rehabilitation of the deck. This is particularly true where studded tires are permitted or where there is a high density of trucks running high tire pressures. Differential wear in the wheel tracks can cause water to pond and accelerate deterioration of the concrete deck as well as decreased safety for the traveling public. Also, surface wear can cause polishing of the deck aggregate at the surface. This in turn reduced the skid resistance of the deck and may require deck rehabilitation for safety/skid resistance purposes.

Spalling. Spalling results from corrosion of the deck reinforcing steel, and is by far the major source of damage/deterioration of concrete bridge decks. Obviously, to reduce the potential for spalling, one should:

1. Reduce the potential for deck surface water and deicing chemicals to get to the rebar.
2. Improve the corrosion resistance characteristics of the rebar.

Reducing concrete cracking, increasing concrete quality, increasing the depth of concrete cover over the rebar, and employing a surface membrane are ways to achieve (1) above. Using epoxy coated rebar or employing a cathodic protection system are ways to achieve item (2).

Each of the above common types of bridge deck deterioration along with recommended mitigation actions are summarized in Table 2.1.

Table 2.1.
Bridge Deck Deterioration
and Corrective Actions

Primary Types of Deterioration	Description	Action to Mitigate Deterioration
Cracking	A basic characteristic of concrete because of its low tensile strength. Cracks occur due to initial drying and shrinking, loadings, changes in humidity/wetness, and seasonal/daily temperature changes.	Use high quality concrete, 2"-2 1/2" of concrete cover and low strength steel rebar, minimize span length and angle of skew, and use simple span construction where possible.
Scaling	A surface phenomenon in which the concrete mortar at the surface flakes due to frost action.	Use low water-cement ratio and air-entrained concrete, and proper care in finishing the concrete.
Wear/Polishing	Differential wear and/or polishing of concrete aggregate in the wheel track due to microscopic crushing of deck material under high tire pressures and rolling friction.	Use hard aggregate and concrete, use angular aggregate, consolidate and finish concrete correctly, and control vehicle tire pressures and use of studded tires.
Spalling	Delamination and/or spalling off of chunks of concrete (normally off the deck) due to rebar corrosion.	Increase rebar concrete cover to 2 1/2" for top of deck, use epoxy coated rebar in top mat, use high quality and nonporous concrete and exercie proper care in placing, consolidating and curing concrete.

The deterioration of bridge decks is mainly attributed to the decks functions which are [17]

- Provide a riding surface for traveling vehicles.

- Transmit live loads to the structural elements of the superstructure and the substructure.

- Protect the superstructure and substructure from the harsh elements contained within the bridges service environment.

In terms of the sources/causes of bridge deck deterioration, four of the primary factors are:

- Faulty Joints
- Improper Drainage
- Rebar Corrosion
- Impact Loading and Live Loads

Faulty Joints. Faulty joints are probably the greatest source of bridge deterioration and premature failure because of leakage of water or worse yet, the leakage of deicing salt water. The photos in Figs. 2.5 and 2.6 were extracted from Reference 17, and show substantial to massive deteriorations resulting from joint leakage. Faulty joints also tend to have adverse effects on the vehicle riding surface, deck alignment, and structural elements within their vicinity. Another problem with deck joints is that in many cases they are restricted from performing their main function, which is to allow for expansions and contractions due to moisture and thermal fluctuations. Expansions may become limited due to the build-up of debris within the joint itself. The accumulation of dirt, rocks, etc. leads to the clogging of joints. When joints become clogged and expansion takes place, internal stresses are transmitted directly through the joint as if the joint did not exist. Clogged joints also result in the transverse shifting of deck segments which in turn cause damage to elements of the superstructure.

In preventing the problems brought on by clogged joints, periodic maintenance is a must. It is important that joints be cleaned regularly, and particularly after the winter months to ensure that accumulated deicing sand is removed from the joints prior to the summer

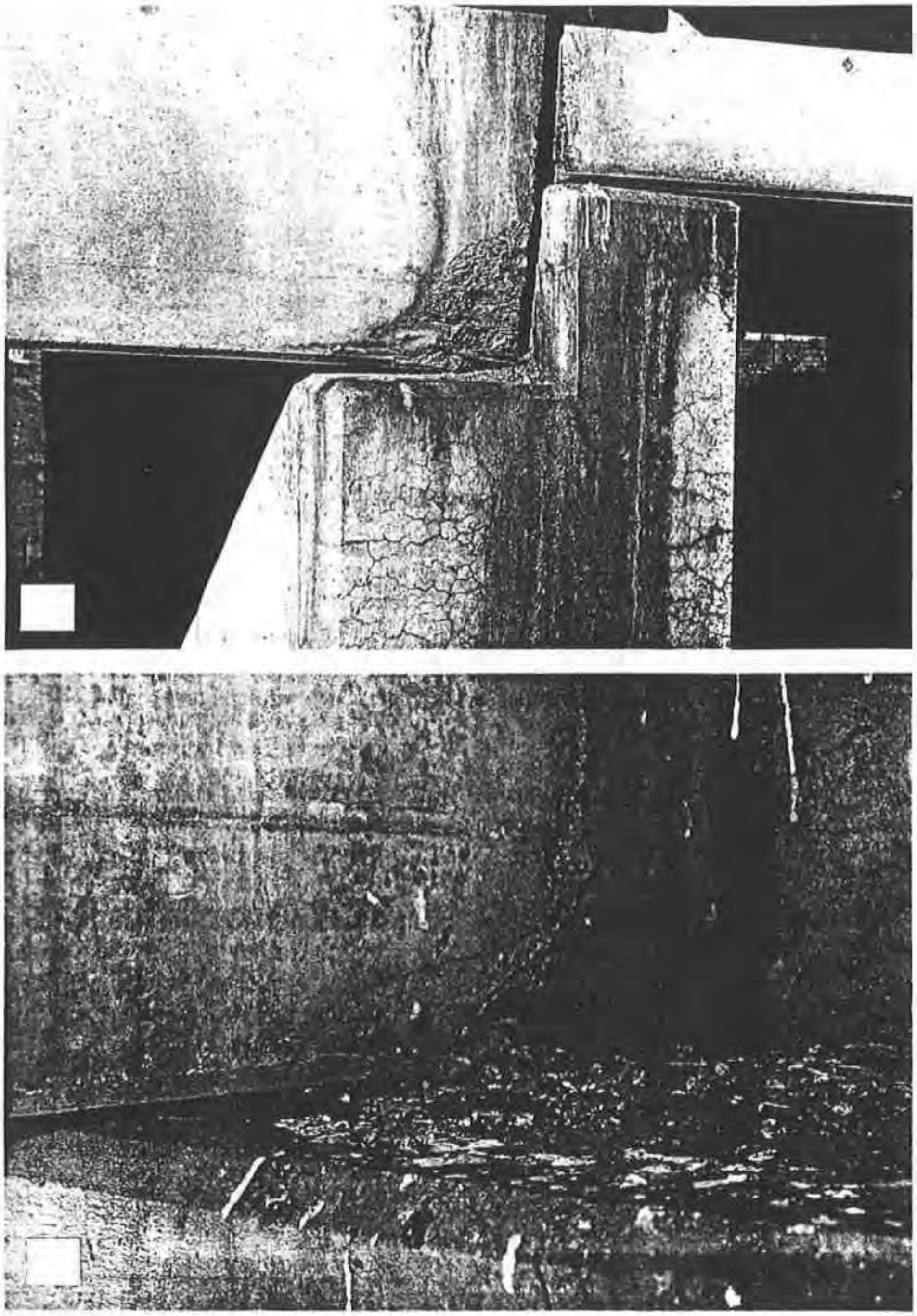


Figure 2.5 Concrete Deteriorations at Beam Ends at Pier and Abutment Due to Water Leakage Through Faulty Joints [17].

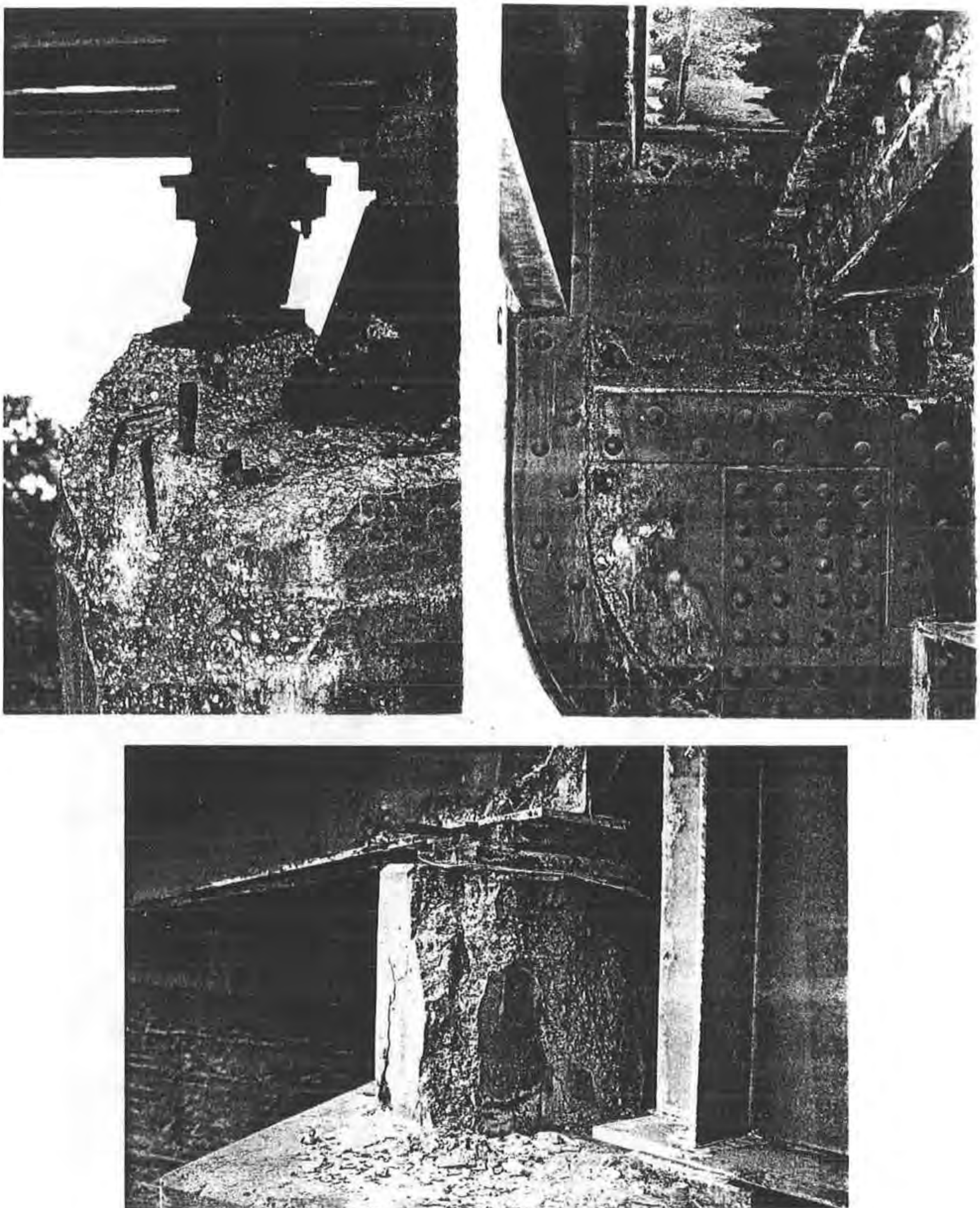


Figure 2.6 Massive Concrete and Steel Deteriorations Due to Water Leakage Through Joints [17].

heating/ expansion season.

As a design factor, joint seals aid in preventing the build-up of debris within the expansion joints. Seals also make maintenance practices much easier and less time consuming. There are a wide variety of seals available on the market today, however neoprene compression seals such as shown in Fig. 2.7 are probably the most popular. Since joints tend to be closed during the summer months, installation of such seals is much easier during the winter. Loosening and pop up of one of the angle irons is sometimes a problem with these joints. This problem is initiated by the constant pounding and impact of vehicle traffic.

Finger joints (see Fig. 2.8) allow for total movements ranging from 4 inches to 24 inches which exceeds that of the open joints. Although such joints can withstand movements up to 24 inches, their durability is increased when the joints range of motion is limited. To prevent the build-up of debris in finger joints, a trough system should be installed to carry away sediment. When installing these troughs, it is important that they are at a slope of 1 inch per foot to ensure a self cleaning action due to water flow [17]. The most common and durable material for such troughs is neoprene which is material used in compression seals mentioned earlier. Finger joints have performed well to date.

<u>Improper Drainage</u>. Proper deck drainage refers to the decks ability to remove quantities of water from its riding surface and discharge it in a manner so that it is not emptied on the concrete and steel members below. Drainage can be retarded due to a variety of factors which include:

- Small inaccuracies in the deck finish
- Shallow slopes and crowns
- Curb confinement

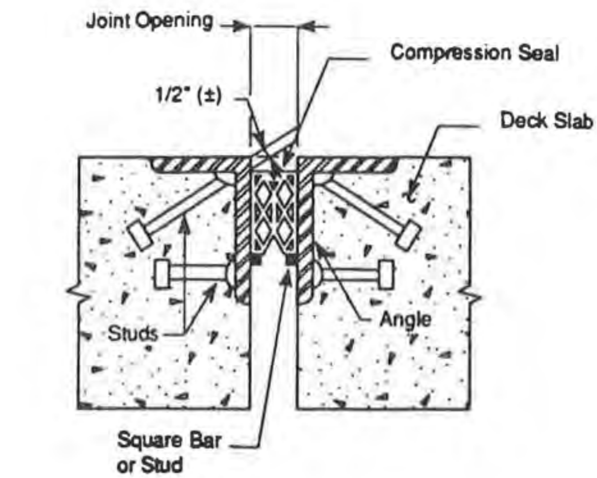


Figure 2.7 Neoprene Compression Seal

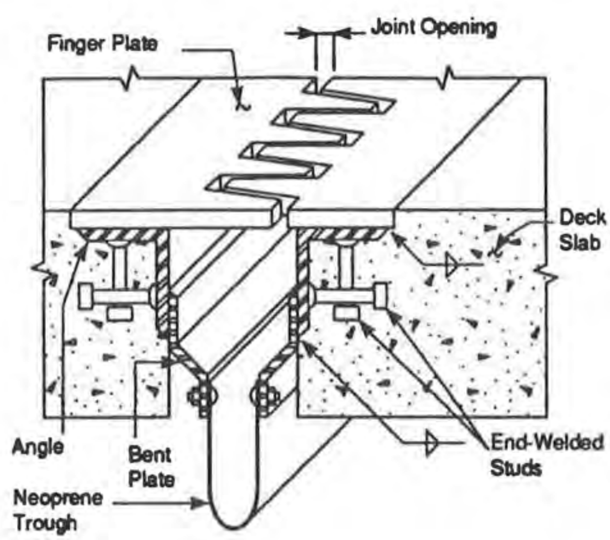


Figure 2.8 Finger Joint

- Improper positioning and/or number of collection devices

- Accumulation of debris in drainage system.

When drainage runoff is emptied onto elements of the superstructure and substructure, significant damages may result. In many bridge decks, water is collected and discharged freely into the air below. When blown by winds, the flow of this discharge is redirected and in many cases comes in contact with structural elements of the super and substructures. Drainage of this sort leads to standing water on elements directly beneath the deck. These elements are normally shaded from the sun, and the standing water leads to scaling, delaminations, and spalling. The problem is often intensified by the confinement of water in joints, bent caps, lower flanges of girders, and any other horizontal surface that prevents runoff.

When designing deck drainage systems, there should be a common practice of connecting discharge piping to the collection devices within the deck. The purpose of these drainage pipes is to transport water from the collection devices to the ground.

Rebar Corrosion. Rebar corrosion can be major problems in concrete bridge decks. As mentioned earlier, the corrosion of rebar is due to moisture and chemical attack. In reducing problems related to rebar corrosion there are several design factors to keep in mind. These include:

- Concrete Cover - The main problem in many bridge decks is the lack of concrete cover between the deck surface and the outermost reinforcement steel. It is recommended that there be at least 2-2 1/2 inches of cover on the upper surface of the deck and one inch on the lower surface. These distances should be measured from the outer most reinforcement bars to the surface of the deck.

- Epoxy Coated Rebar "Green Steel" - The use of epoxy coated steel has significantly reduced problems with corrosion. Such coatings are somewhat brittle and tend to break away on areas where rebar has been bent or shaped. To prevent corrosion in these areas, the chips should be touched up with an epoxy coating that is similar in nature to that of the original.

- Concrete Density - The use of the air entrainment in bridge decks helps increase the concretes density by decreasing the water/cement ratio. This increase in density makes the concrete less permeable to moisture and harmful solutions. It is important that the entrained air voids are not interconnected. Under such conditions the concretes permeability is actually increased which leads to accelerated corrosion of the reinforcement steel. Table 2.2 gives suggested air entrainment percentages for concrete deck mixtures.

Table 2.2
Recommended Air Contents For Bridge Deck Concrete* [3].

Maximum size of coarse aggregate, In.	Air content* percent by volume
1 1/2	5.5 ± 1
1	6 ± 1
3/4	7 ± 1
1/2	8 ± 1

*Where no deicers are used, these may be reduced approximately 1.5 to 2 percent.

When considering the service environments for reinforced concrete decks they can be divided into three categories [47]:

- Normal Urban Environment
- Heavily Polluted Industrial Environment
- Marine Environment

As a rule of thumb, in increasing order these different type environments require increased concrete cover, with the thickness being in a marine environment.

Impact Loading and Live Loads. Heavier and more frequent applications of truck loads are being applied to our bridges than ever before. Additionally, they are being applied through trucks running much larger tire pressure than ever before. The bridge element that these loads are being directly applied to is the deck. These facts coupled with significant impact effects due to possible bridge joint problems and deck spalling and pot hole problems lead to large localized and dynamic loads on bridge decks. The net effect of this is accelerated deterioration of the top surface of our bridge decks. In turn this results in a rougher deck surface and hence greater impact loadings and further acceleration of deck deterioration.

To minimize the above effects, efforts should be made to

- Maintain a smooth riding surface (this would include minimizing the number of deck joints)
- Maintain deck surface in a good condition of repair
- Use highest quality concrete practical for bridge decks
- Police truck traffic with weight/tire pressure check stations.

Obviously, careful planning and design, proper selection of materials, good construction practices, and good maintenance practices are all essential to achieve durable bridge decks. Some specific practices or guidelines in each of these areas to improve bridge deck durability are presented below.

Design Practices. Durability, in general, is similar to one of its components, i.e., fatigue, in that the initiations of the deterioration is a highly localized problem. As with fatigue problems (which is a durability issue), common sources of nondurability often occur due to a lack of attention to detail or to inappropriate detailing practices. Some recommended design practices to enhance the durability performance of concrete bridge decks are given below.

1. Employ thicker decks.
2. Employ large cover (approximately 2 1/2") over top rebar.
3. Employ highest quality concrete in the deck, i.e., low water-cement ratio, high strength, air-entrained and low permeability.
4. Employ epoxy coated rebar in top layer of the deck.

5. Employ shorter spans, simple spans and zero skew angle where practical to minimize incidences of deck cracking.

6. Employ good deck drainage to get water off of the deck (and away from the bridge). A small number of large/nonclogging drains are preferable to many smaller drains.

7. Minimize the number of joints in the deck (these are sources of water damage to components below the deck and are subject to clogging and causing deck and bridge distress).

8. Place a paramount importance on designing joint for inspectability and maintainability for those expansion/contraction joints that are required.

Selection of Material. Selection of the bridge materials is a decision that has a major impact on the durability and longevity of that bridge. For reinforced concrete bridges this means selection of the concrete and the reinforcing steel.

Durable concrete requires good quality raw materials that are clean prior to mixing and that are blended in a good mixture design with a low water-cement ratio and with air entrainment for freeze-thaw resistance. This will result in a high strength and low porosity concrete. Additives to inhibit corrosion of steel rebar should be considered; however there appears to be more R&D work needed in this area before specific recommendations can be made.

Durable steel rebar is rebar that will not corrode in its encapsulated environment in the concrete bridge. Regular uncoated steel will corrode in such an environment to differing degrees depending on the bridge environment, quality of concrete and bar cover. It should be noted that high stress/strain levels in bridge rebar imply high deformations and cracking of the adjacent concrete. This in turn increases porosity and promotes corrosion. Hence lower strength-grades of steel rebar are preferable from a durability viewpoint.

Stainless-steel-clad rebars have been experimented with as have other metallic coatings such as zinc and cadmium. Epoxy coated rebar are quite popular for use in bridge decks, and information to date indicates that they are performing well. Rebar composites of fiberglass are currently being researched. If a noncorroding reinforcing bar can be developed, it will lead to a significant enhancement in the life/durability of concrete bridge decks. Currently, the corrosion of steel rebar is probably the single greatest cause of concrete bridge deck deterioration or reduced durability.

It should be noted that bridge superstructure concrete generally account for approximately 23 percent of the total bridge concrete, but the superstructure concrete accounts for approximately 99 percent of the bridge concrete maintenance cost [49]. Based on this, one could speculate that deck concrete accounts for approximately half of the superstructure concrete or about 12 percent of the total bridge concrete and probably 80-90 percent of the bridge concrete maintenance cost. Hence extra attention to the bridge deck seems appropriate in order to achieve improved cost effectiveness and improved bridge durability/longevity. These data seem to indicate that as high a quality concrete as is realistically feasible should be used for bridge decks, and the top deck mat should be epoxy coated mild steel rebar.

Construction Practices. Good construction practices in all areas, i.e., preconstruction planning, inspection, formwork, reinforcement placement and supports, and concrete placement, consolidation, finishing and curing, are essential to achieve a durable concrete bridge. To have the greatest positive impact on bridge durability, construction practices should focus on the following two items.

1. The quality of the in-place concrete in the bridge and particularly that of the bridge deck.

2. The cover over the reinforcing steel and particularly the top rebar mat in the deck.

The first item above requires good quality materials, a good mix design, a low water-cement ratio and air entrainment. It requires placement under good environmental conditions both for the effect that these conditions have on the material characteristics, as well as on the quality of workmanship of the workers and in turn on the quality of their finished product. It also requires proper consolidation and finishing work and good moist curing conditions. Regarding durability, the in-place quality of concrete in a bridge is strongly related to its porosity. This in turn is controlled by the mix design water-cement ratio, the degree of consolidation, and the design of hydration (which depends on the curing process and the age of the concrete).

The second item relates to mitigation of corrosion of the top steel and thus mitigation of concrete spalling. It is essential that construction practices are such that the specified rebar cover is achieved. The rebar must be accurately placed and firmly seated/secured against movement during placement of the deck concrete. Consideration should be given in the concrete finishing operations to the finishing machine to be employed, and to the rail and rail supports for the machine so that excessive deflections of the rails do not result in the specified top rebar cover being violated. Evaluations of existing bridge decks reveal many cases where actual covers are significantly less than that which was specified.

It should be noted that the Kansas DOT estimated in a report [36] that by increasing the concrete cover from 2 inches to 3 inches, and by decreasing the water-cement ratio of concrete from 0.44 to 0.35, the life a concrete deck could be tripled.

<u>Maintenance Practices</u>. Primarily, durability must be designed and constructed into bridge decks. If this is not done, maintenance personnel are fighting a loosing battle. However, a well designed and constructed bridge can and will deteriorate prematurely if not properly maintained. Some recommended maintenance practices to enhance bridge deck durability are given below.

1. In winter operations, get deicing salts and sand off bridges as quickly as feasible.
2. Disallow the use of studded tires.

3. Keep all deck joints and drains clean and clear of debris.

4. Prevent excessive truck loads and tire pressures from getting on bridges by stricter enforcement of the laws via weight/tire pressure stations.

5. Place an emphasis on bridge durability/longevity in periodic inspections to detect abnormal deteriorations early.

6. Conduct more frequent inspections of bridge decks since this is the location of most bridge deteriorations.

7. Make needed repairs in a timely and permanent manner to prevent penetration of water into the deck.

Because of their unique physical characteristics, loading, and exposures, Austin [9] states that bridge decks are subjected to accelerated deteriorations. He indicated that thin deck elements are more vulnerable to repeated exposures to freezing and thawing, as well as wet-dry cycles than other structures, and are also more susceptible to rebar corrosion. The main factors influencing rebar corrosion, as indicated earlier, are; (1) thickness of concrete cover, (2) corrosiveness of environment, and (3) alkalinity, permeability, and quality of the concrete. Inadequate cover (less than 2") has been cited as the largest single cause of spalling and deterioration of concrete bridge decks.

Austin [9] indicated that many premature deck "failure" problems can be attributed to the initial design and construction practices, such as:

1. Inadequate design and/or installation of expansion joints. Expansion joints should be able to accommodate the significant temperature fluctuations and load transfer concerns unique to thin, highly exposed bridge decks.

2. Inadequate design and/or installation of drainage. Inadequate drainage of bridge decks results in adverse ponding of water and de-icing chemicals.

3. Use of high slump concrete (i.e., high w/c ratios) to attain adequate workability in highly reinforced bridge decks. This results in decks which are more susceptible to

scaling, dusting, thermal cracking, plastic settlement cracking, and other flaws. Additionally, higher w/c ratio concretes contribute to high rates of corrosion in reinforcement, since aggressive chloride ions are more likely to permeate through the more extensive pore water structure.

4. Mix designs with higher paste contents have a greater tendency to deteriorate. Excessive cement contents will generate extra heat through hydration which may produce thermal cracking. Use of absorptive coarse aggregate can later result in pop-outs and spalls. Mix designs with inadequate air entrainment are also susceptible to deterioration.

5. Improper curing. Premature curing or early termination of curing will permit increased shrinkage at a time when the concrete has low strength. Improper curing will result in reduced durability. Dry hot winds during placement may adversely affect the quality of the concrete.

6. Improper vibration. Concrete may have been subjected to excessive vibration resulting in a segregation of the mix or inadequate vibration resulting in honeycombing.

7. Poor finishing techniques can significantly contribute to plastic shrinkage cracking in thin slabs. Pavements subject to a hasty screeding will exhibit more cracking than those screeded at a slower rate.

8. Poor form construction (i.e., inadequate shoring) may result in premature loading of recently placed concrete. Additionally, poor joint construction may result in a non-functional joint. Inadequate inspection and consequently limited maintenance is also a major factor in the deterioration of bridge decks. Accumulations of dirt and debris in expansion joints and drains as shown in Fig. 2.9 can markedly

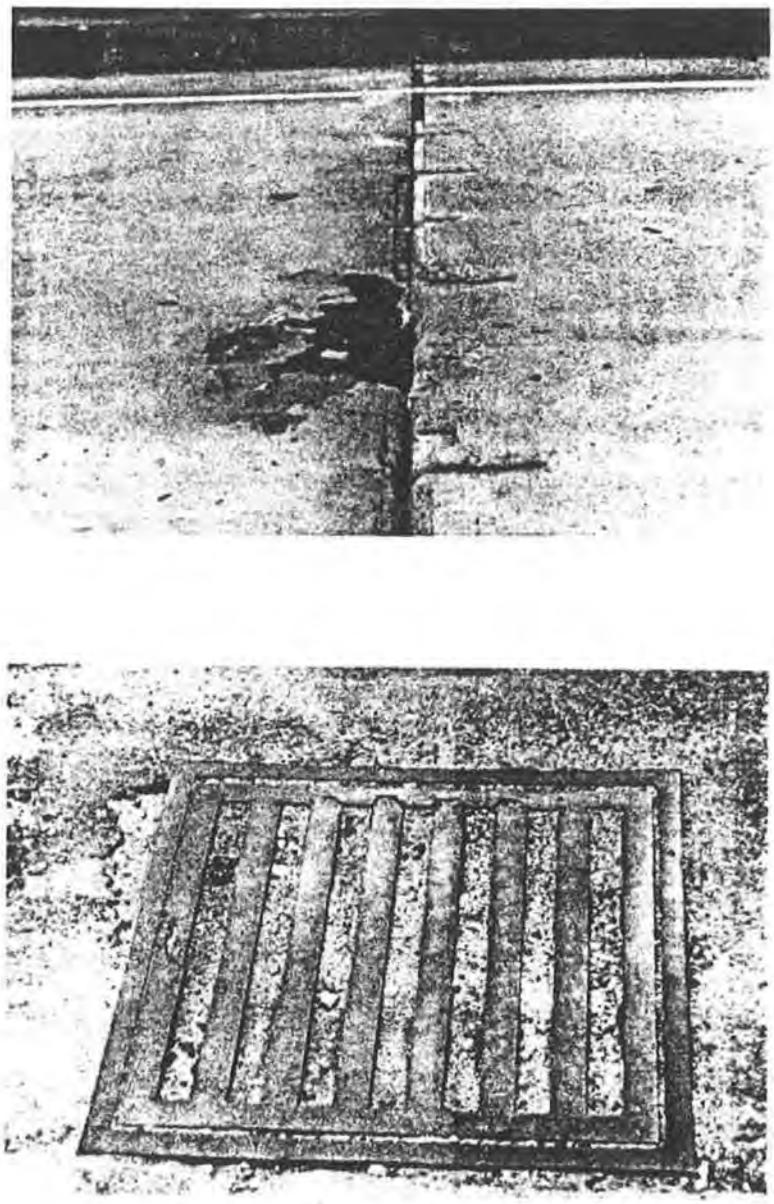


Figure 2.9 Dirt and Debris Filled Joint and Drain [16]

compromise the best of designs.

A number of studies have been conducted at state and national levels to determine the primary reasons for the relatively rapid rate of bridge deck deteriorations. The major findings of these studies are summarized below.

1. The rising rate of deicer usage by many highway departments (an increase of 400% in chloride slats usage during the decade of the sixties) is felt to be a major cause of deck deteriorations.
2. Environmental factors (primarily freeze-thaw cycles) were felt to be a significant cause of bride deck deterioration.
3. Moisture in environmentally caused deteriorations (freeze-thaw) and as a carrier of deicers is a major factor in deteriorations. Poorly drained decks deteriorated more rapidly than well drained decks.
4. Shallow reinforcement is a major contributor to deteriorations. Longitudinal and transverse cracks tend to follow the reinforcing steel where cover is small.
5. Corrosion of reinforcement is a major cause of fracture planes and deck spalls which in turn are the major type of deck deterioration.
6. Several construction related problems have led to premature deck deteriorating. These include, insufficient bar cover, improper concrete finishing (overworking and overwetting), improper curing and drying procedures, pouring of deck in extreme weather, i.e., hot, dry and windy weather or cold weather.
7. Bridge deck durability is directly related to the quality of the deck concrete.
8. In Kansas, greater deck deteriorations were observed on steel superstructures than on concrete superstructures.

9. Continuous structures were found in some studies to have a higher frequency of deck cracks and spalls, especially in areas of negative moment, than simple supported bridges.
10. Thin deck slabs were found to have more cracks and spalls than thick slabs.
11. Increased loads and ADT are important factors in deck deterioration.
12. Results of a Penn DOT survey of 249 four-year old Pennsylvania bridge decks are shown in Table 2.3. In this table, the form or type of deterioration decrease in severity of deterioration or importance from left to right, and the causes of each type of deterioration decrease in frequency or importance from top to bottom.

Table 2.3
Relationships Between the Major Forms of Deck Deterioration and the Factors Causing Them [5].

Fracture Planes and Spalls	Cracks	Surface Mortar Deterioration	Miscellaneous Forms of Deterioration
Depth of steel	Construction practices	Construction practices (especially finishing)	Difference in elevation of approach slab and deck as related to broken slab edges
Superstructure type	Form type	Antiskid material (may be related to use of $CaCl_2$ on stock piles)	Aggregate sources as related to popouts
Construction practices	Span length	Form type	Snowplow damage as related to parapet
Flexural strength	Superstructure type	Average daily traffic	
Retarder use	Maximum placement temperature	Deicing chemical type	
No. of salt applications per year		Finishing machine	

2.5 Bridge Deteriorations and Durability

Some of the primary reasons for deterioration of bridge components and subcomponents cited in the literature are:

- Deicing compounds (not applicable to Alabama)
- Heavy truck traffic (high volume of truck traffic and overweight trucks)
- Lack of funding
- Flooding/Scouring

Many states use winter road-salting operations or are located near some form of seawater that will form a harmful sodium chloride solution on bridges. This solution will corrode and weaken the steel elements by direct contact or by penetrating through the concrete. With current specifications emphasizing concrete strength rather than impermeability, and with the increased annual usage of deicing salts, there is probably a greater chance today of concrete damage from this source than ever before. Figure 2.10 shows the typical attack zones of salts, and Figure 2.11 shows areas that should be waterproofed to provide protection from saltwater solutions [42].

Heavy truck traffic can cause massive deterioration for bridges. Bridge engineers believe that increasing number and severity of overload cycles like those caused by an overweight vehicle accelerates deterioration of the bridge. Also, these engineers estimate that the bouncing of vehicles caused by potholes etc., imposes an extra load on the bridge up to 30% of the truck's weight. So, some feel there should be a speed reduction for heavier trucks crossing bridges to help prevent deterioration of the bridge's components [26].

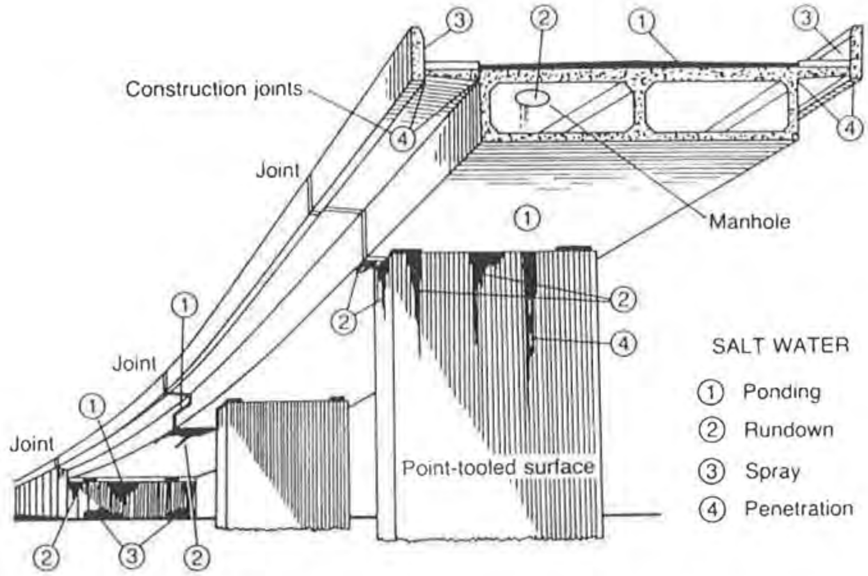


Figure 2.10 Typical Salt Attack Zones [42].

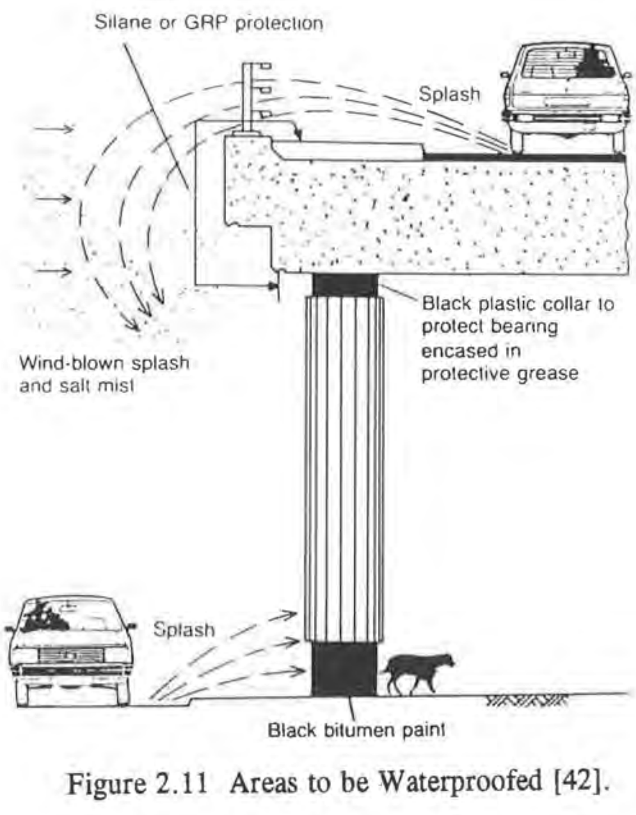


Figure 2.11 Areas to be Waterproofed [42].

Some engineers respond to lack of funding problems by designing and building substandard bridges [26]. When one looks at the overall life-cycle cost of a bridge and recognizes the cost of constant repairs and maintenances on inferior spans and components, it is clear that cheaper initial cost does not mean life-cycle cost savings.

Reference [12] indicates that more bridges fail in floods than in any other way. Much of the damage associated with floods is due to scour problems. For new bridges, this problem is probably best solved by deepening the foundations to accommodate the scour predicted in the worst cases. This will only require extra length of piles or caissons or deeper spread footings. For old bridges, the following offer good solution alternatives [12].

1. Use riprap at the level of the bottom of the deepest (future) scour hole for which the present foundation is adequate.
2. Use spur dikes to move the scour hole away from vulnerable abutments.
3. Make channel improvements to improve the hydraulics of the bridge opening.
4. Use additions to the present foundation (such as a sheet pile ring).

Reflective cracking above PCC pavement joints is a problem that afflicts countless overlays, including bridge overlays. Since sealing of these cracks is inevitable, a logical approach, according to the Asphalt Institute, is to rout and seal them at the initial time of overlay construction, thus preventing further breakdown of the pavement surface [44]. Sawcut and scaling of the asphalt overlay is an effective technique to reduce the detrimental effects of reflective cracking at the joints and extend the service life of the overlay. The technique establishes a weakened plane joint in the overlay directly above the existing joint.

The Asphalt Institute breaks the sawcut and seal technique down to five procedural steps [44]:

- Locating and referencing existing joints in the PCC slab

- Placing the overlay
- Sawcutting directly above the referenced joint
- Cleaning and drying the sawcut, and
- Sealing the sawcut.

FHWA calls locating the transverse PCC joint the most critical step in sawing and sealing the overlay. Experience has shown that as little as one inch deviation from the existing joint location can cause the joint to reflect through the overlay at its location rather than at the sawed joint.

New York transportation authorities found that on longer slabs the joint sealer had pulled loose from the joint face, but the joints themselves had remained in good condition. The sealer failures were due to their insufficient width in relation to slab length and the related horizontal movement which required elongations greater than the material's capacity. As a result, New York has adopted into their specification a table varying the width and depth of sawed joint based on the slab length. New York also specifies that a sawcut 1/8 in. wide and 2 1/2 in. deep be included where the overlay thickness exceeds 4 1/2 in. to ensure cracks form between the sawcut and the existing joint on thicker overlays [44].

There are several ways of increasing durability and to extend effective service life of a bridge. Some of these are listed below [29].

1. During planning stage.
   - Chose design by looking at the life-cycle cost
   - Make decisions based on the environment (weather or traffic)
2. During design stage.
   - Recognize this stage has greatest impact on quality and design accordingly

- Chose material and details that are best and economical
- Add redundancy so if one component fails the other will do the job
- State the design accurately and clearly so as not to compromise durability and quality

3. During construction stage.
   - Use prefabricated sections (for quality control)
   - Hire a "quality conscious" contractor
   - Require technical qualifications to bidder's level of experience

Good design practice typically produces several initial designs using different span arrangements and various relevant construction materials and techniques. After an initial 'coarse sieving' of the differing proposals, the choice is usually reduced to two alternatives using judgements based on the desirable features of [42]:

- Design and construction economy in both time and money
- Good appearance
- Ride quality and safety
- Ease of access for inspection and maintenance
- Design life of at least 120 years in the UK, with a minimum of maintenance
- Ready repeatability and standardization for multi-span or multi-bridge applications
- Ease of repair, possible strengthening and possible modification or replacement during roads widening.

A 'finer sieving' procedure will usually produce a preferred alternative, which can then be recommended to the client.

In such a procedure, it is clear that different weighing can be placing on each of the desirable features. In most instances, however, the high cost of construction tends to dominate the choice between alternatives, with the estimated periodic costs of maintenance, and ease of access, repair, and strengthening, modification or replacement only recently gaining importance in the overall cost considerations.

Fortunately bridge designers are beginning to pay more attention to bridge durability and whole-life or life-cycle costs, defined as 'the costs of all activities associated with a bridge during its life.' For example in the UK, a bridge's design life is currently set at 120 years. In that time-span, the significant cost stages are [42]:

- The high initial cost of design and construction
- The regular inspection and maintenance costs over the bridge lifetime
- The repairs which could be expected during the lifetime, cost of which include those relating to traffic disruption
- The possibility of one or more strengthening operations to cater for increased traffic loading or design code changes, with again the costs of possible traffic disruption
- The possibility of bridge modification or replacement on-line due to widening of the road carried or crossed, with even greater costs of possible traffic disruption.

The durability of the newly designed and constructed bridge and the resulting economy of maintenance is generally the second most important component (initial cost is the most important) of the whole-life cost. It is encouraging that more owners are seriously examining the benefits of paying extra in first cost for a more durable bridge [42].

2.6 Jointless and Continuous Deck Bridges

Traditionally, a system of expansion joints, roller supports, and other structural releases has been provided on long multiple-span highway bridges to permit thermal expansion and contraction. The desirable characteristics of an expansion joint system are water tightness, smooth ridability, low noise level, and wear resistance. In reality, however, the performance of many joint systems is disappointing. When subjected to traffic and bridge movement, they fail in one or more important aspects, notably water tightness. Flow of runoff water through open bridge deck joints or leaking sealed joints has been one of the major causes of extensive maintenance and costly rehabilitation work on bridges in general.

Critical substructural elements that are commonly damaged by water runoff through expansion joints include steel girders and stringers, bearings, and rollers. Corrosion in this critical area is particularly severe for weathering steel bridges that have no protective coatings. Similarly, the coating system of painted bridges that are fabricated from nonatmospheric corrosion-resistant steels deteriorates much faster near bridge expansion joints. In addition, reinforced concrete structural members, such as abutments, piers and pier caps, are often subjected to scaling and spalling caused by deck runoff through joints, which subsequently leads to corrosion of exposed reinforcing steel.

In some bridges, troughs have been placed below open expansion joints to collect the runoff water and discharge it through drain pipes away from the structure. This solution does not seem to be viable because it introduces an additional item to clean and maintain. The original problem reoccurs as soon as accumulated roadway debris clogs the troughs and pipes, causing the runoff water to overflow.

For bridges with reinforced concrete decks, sections of the deck located adjacent to expansion joints are particularly vulnerable to deterioration. Permeable stress cracks in the concrete, developed from repetitive traffic pounding at the joint or edge distress due to the cantilever action of finger-type joints, leave reinforcing steel exposed to water and corrosive chlorides.

There has been a move to eliminate expansion joints from highway bridges altogether. This move has resulted in the development of jointless bridge concepts that serve to accomplish the following desirable design objectives: (1) long-term serviceability of the structure; (2) minimal maintenance requirements; (3) economical construction; and (4) improved overall performance of the facility [7]. A review of the performance of several jointless bridges indicates that these structures are generally performing exactly as intended. In most cases, the elimination of expansion joints has not caused major structural damage or affected the long-term serviceability of these structures. These bridges appear to be popular with the states that build them due to reduced construction and maintenance costs. Many of the problems encountered during the earlier development phase have been reduced or eliminated [7].

For longer jointless bridges in the range of 450 ft or more (137.3 m), several states have encountered problems in the approach slab/backfill areas. Common complaints concern the settlement of the approach slabs and the formation of bumps at the bridge ends. For some states, such as Tennessee, maintenance of the approach slab is an acceptable trade-off to the more extensive problems associated with bridge expansion joints. Other states, such as California, have redesigned their approach-slab systems [7].

Deck continuity and the minimizing of deck joints is undoubtedly one of the key factors contributing to the durability and ease of maintenance of both bridge decks and substructures. This is evident in comparing the relative damage due to winter de-icing salt between a simply supported flyover and a continuance flyover of about the same age shown in Fig. 2.12 [42]. Five main advantages to be gained by using multi-span deck continuity are illustrated in Fig 2.13 and are listed below [42].

1. Elimination of deck and parapet joints at the piers. This represents savings in both construction and maintenance costs-maintenance which includes that of the underlying piers. Elimination of joints also gives the vehicles crossing the bridge a better ride.

2. Saving in deck depth. Reduced deck depth can reduce construction cost of both the deck and the supporting substructure, and can also reduce the cost of associated approach ramps or earthworks. The latter saving is probably the most significant construction-cost economy.

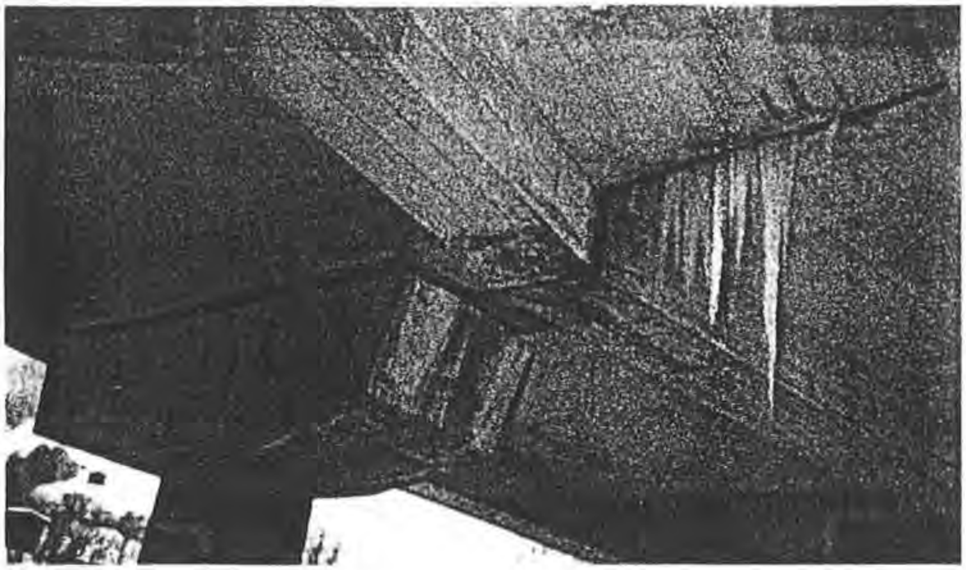
3. Replacing two rows of bearings by a single row of central bearings. The piers of crossheads can be reduced in thickness not only because the single row of bearings takes up less room at the top, but because the deck dead load has reduced and the live load moments applied by off-centre pairs of bearings are removed. Significant cost savings in pier foundations also result. Additional savings occur because only half the number of bearings are needed at each pier. Although the bearings will be of higher capacity, the reduced number will result in cost savings.

4. Full advantage of haunched girders can be utilized. This will result in more efficient use of materials and in cost savings.

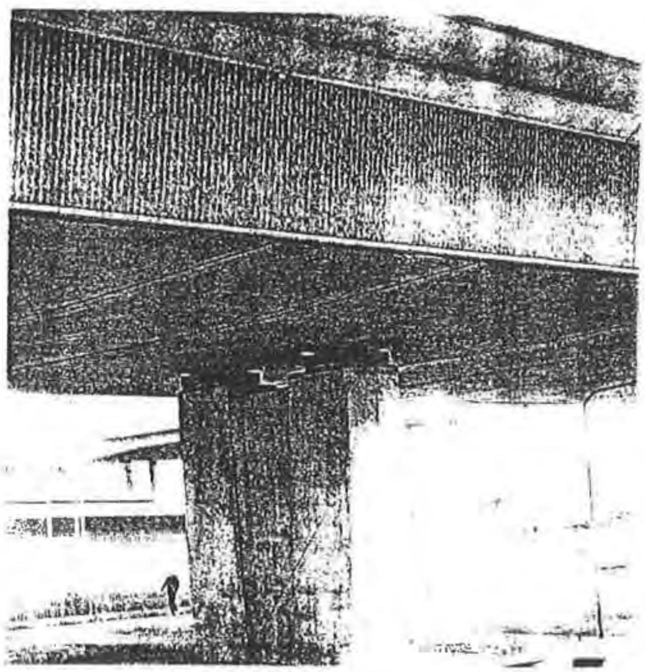
5. Continuity means extra deck redundancy. Redundancy generally guards against sudden collapse in overload situations.

It should be noted that a main disadvantage to continuous bridge construction is that daily heating cycles cause significant thermal stresses. These stresses do not influence the strength of the superstructure; however, they probably do significantly affect long-term serviceability or durability/longevity of the bridges.

Preferably the superstructure should be designed to rest on short pedestals at the bearing locations and the pier/bent caps and abutments designed to be without horizontal surfaces on which water can pool. The bearings should be as simple, maintenance free and durable as feasible. Elastomeric bearing pads seem to do an excellent job of meeting those desirable characteristics.

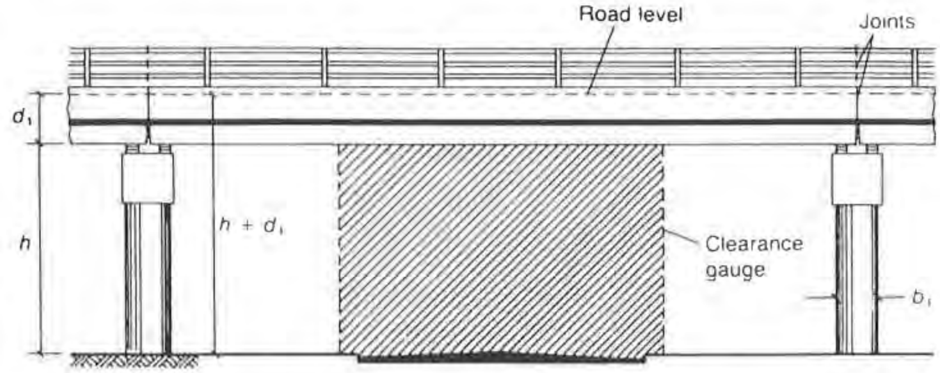


a) Simply Supported Flyover

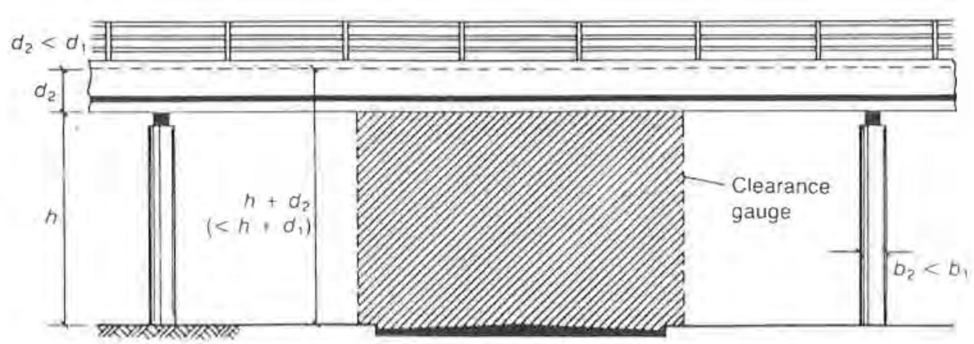


b) Continuous Flyover

Figure 2.12 Simply Supported and Continuous Flyovers of Approximately the Same Age in London [42].



a) Simply supported superstructure



b) Continuous superstructure
Figure 2.13 Advantages of Superstructure Continuity [42].

2.7 Bridge Material Durability

To build durable bridges, it is essential to use durable materials. Modern highway bridges are constructed of steel, reinforced concrete, prestressed concrete, or some combination of these materials. Hence, only the factors affecting the failures, deteriorations and durabilities of these materials are considered in this study.

2.7.1 Concrete

Durable concrete begins with quality and compatible material ingredients. That is, stable and strong materials that are chemically and thermally compatible with each other and the other materials with which they will be in contact. Durability of hydraulic-cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment.

Reference 21 points out that a factor which typically receives too little consideration in the design of concrete mixtures is durability. This same reference cites that the affect of weather in the deterioration of concrete structures is due, in part, to the cyclic expansion and contraction under changing moisture and temperature conditions. In part deteriorations are due to the expansive forces of ice crystals as they are formed in the pores of the concrete, and in part to the leaching of soluble compounds from the mass by water. It follows, therefore, that a relatively watertight concrete is a more weather-resistant concrete.

Reference 22 lists many factors which cause deterioration of concrete. Condensed discussions (from [22]) of these factors are presented below.

Poor Design Details. Some of the most common design deficiencies that lead to premature concrete bridge deterioration are not specifying very high quality concrete, i.e., low water-cement ratio, air entrainment and appropriate admixtures to make the concrete as impermeable and durable, and yet as workable as possible. Other design deficiencies include insufficient cover over rebar or prestressing wires, insufficient attention to drainage and scupper sizes and locations (i.e., use of fewer large scuppers and make sure they don't discharge on to substructure concrete below), expansion joints and bridge joints in general. It should be recalled that fresh concrete settles in the forms when bleeding and evaporation or drainage of the water takes place. And, the greater the water content of the mix, the greater the settlement. This in turn causes separation on the underside of rebar and prestressing steel and can cause bonding problems as well as corrosion problems.

Construction Deficiencies. Some of the most common construction deficiencies which have a large impact on concrete and bridge durability are:

1. Insufficient rebar cover
2. Insufficient spacing between rebar
3. Adding water to concrete at sight and reducing its quality and durability
4. Improper handling and/or placing of wet concrete
5. Incomplete consolidation of wet concrete
6. Improper finishing of concrete
7. Improper curing of concrete
8. Pouring concrete in too extremes of weather, i.e., too cold or too hot weather
9. Removing forms too early
10. Improper inspection.

Temperature Variations. Freezing and thawing is the most common cause of deterioration in this category. It is not a major problem in Alabama, however, it does occur in north Alabama to some extent. The best means to address this problem is to use air entrained concrete and avoid water sitting on the concrete. Concrete made with good quality aggregates, a low water-cement ratio, and a proper air-void system, which is allowed to develop proper maturity before being exposed to severe freezing and thawing, will be highly resistant to such action.

Bridges are subject to quite a range of solar temperature fluctuations on a daily basis, hence using materials, i.e., cement mortar, aggregates, and steel, with compatible coefficients of thermal expansion is important. Also, restraining concrete members or sections from thermally "breathing" gives rise to inducing thermal stress flucations, cracking and possible premature failure. Like most materials, concrete expands with rising temperatures and contracts with falling ones. The relation is approximately linear and is expressed by the coefficient of thermal expansion. The coefficient of thermal expansion is about 11 MK^{-1}, but it may range from 7 to 12 MK^{-1}. The value depends mostly on the coefficient of the aggregates; the aggregates tend to reduce the coefficient of neat cement, which is about 13.5 MK^{-1}. Fortunately, the normal value for reinforced concrete construction is quite similar to the coefficient for steel.

Chemical Attack. The two primary sources of chemical attack on concrete bridges are:

1. The use of salts or chemical deicing agents (sodium and calcium chloride are the principal salts applied to bridge decks). They cause scaling of the concrete surface, corrosion of embedded steel, and concrete spalling.
2. Chemicals in the soil or water surrounding concrete substructure members like the salt in seawater, for example, are a major source of damage.

The effect of chemical reactions depends on whether the combination occupies a greater or smaller volume than the original constituents. Hydration, the reaction of water with cement, normally results in a volume reduction unless special expansive cements are used. The reactions of high-alkali cements and certain aggregates result in expansion, which may be severe enough to disrupt the concrete, sometimes years after construction.

The best means of protection from chemical attack is to keep the chemicals away from the concrete where possible and to use impermeable quality concrete, good concrete cover thickness, and corrosion protection systems on the steel such as an epoxy coating.

Reactive Aggregates and High-Alkali Cement. Although aggregate is considered to be an inert filler in concrete, such is not always the case. Certain aggregates can react with portland cement, causing expansion and deterioration. Reactive aggregates and high alkali cements cause swelling, map cracking and popouts in concrete. To avoid this potential source of concrete deterioration, one should avoid using reactive aggregates and cements with high alkali content. If this is not possible, then an admixture (such as pozzolan) should be used which will mitigate the reaction. In these cases, special expertise regarding the mix design should be arranged.

Moisture Absorption. Moisture absorption causes concrete to swell and then shrink when it drys which in turn causes microcracking. Additionally, the departing water or seawater may leach out the lime in the cement and leave a weak powdery residue, and/or serve as a source of corrosion of embedded steel. In preventing moisture absorption problems it is important to note that proper drainage cannot be over stressed.

Wear, Abrasion, and Scouring. Wearing and abrasion of the concrete will cause polishing of the concrete which enhances deterioration. The use of studded tires on automobiles has caused serious wear in concrete pavements and bridge decks. High tire pressures also cause early deterioration of concrete pavements and decks. The use of high-quality concrete and, in extreme cases, a very hard aggregate will usually result in adequate durability under these exposures. Scouring is the destruction caused by sand and silts that erode the concrete by wind or hydraulic forces.

Shrinkage and Volume Changes. Although volume changes in concrete are small, they are important because the concrete is normally constrained by itself, steel rebar, abutments and others. Concrete expands with a gain in moisture and contracts with a loss of moisture. As drying takes place, concrete near the surface dries faster than the inner concrete causing tensile stresses and possibly cracking. If motion due to these volume changes is restricted, a series of problems may occur (mainly in bridge deck). When a concrete slab is in its shrinkage phase and motion is restricted, cracks can begin to occur due to the large induced internal stresses. In the expansion phase, limited movement can led to cracking as well as transverse shifting and twisting of deck segments.

When hardened concrete dries, it shrinks, and when it gains moisture again, it expands. Moisture shrinkage is mainly a function of the water-cement ratio and increases with it. For a given water-cement ratio, the greater the proportion of cement paste, the greater the shrinkage. In any case, shrinkage increases with the water content of the mix. As suggested by Fig. 2.14, the relation is nearly linear.

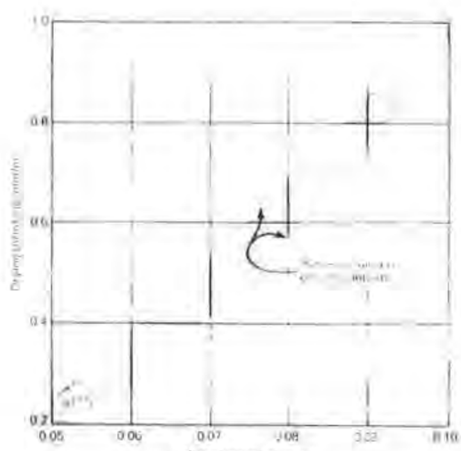


Figure 2.14 Shrinkage vs. Water Content [21].

By keeping the water content of concrete as low as possible, shrinkage can be greatly reduced. The use of low slump mixtures and construction practices that minimize water requirements are both important to the reduction of shrinkage cracking. In addition to a reduced water/cement ratio, the use of hard aggregate can reduce volume change due to shrinkage. It is obviously best to avoid the use of aggregate with high drying shrinkage properties and clay contents. Quartz, granite, feldspar, limestone, and dolomite aggregates generally result in a concrete with low drying shrinkage characteristics [29].

Expansive cement (Types K, M, and S) offer an opportunity to have concrete expand in the moist curing stage to compensate for later shrinkage in the dry curing stage. The concrete mixture can be designed where the two volume changes off-set each other to provide a net volume change of zero. The use of expansive cements to minimize concrete shrinkage cracking is beginning to gain popularity in this country. Such concrete is referred to as shrinkage-compensating concrete.

Collision Damage. These are accident occurrence that can not be predicted, but will cause deterioration of the concrete. Routine inspections would help to find problems that are caused by this damage.

Shock Waves. Deterioration can be caused by shock waves that are formed either by earthquakes or by nearby blast from a construction site. These waves will cause the concrete to crack and then allow water to reach the reinforcement. Also, pile driving will send waves through the concrete pile that will also cause it to destruct.

Fire Damage. If the concrete is subjected to temperatures of 570°F or more the concrete's paste will weaken.

Corrosion of Reinforcement Bars. Corrosion of the reinforcement bars is the electrochemical degradation of steel in the concrete. It occurs when the passivity of the steel is destroyed by either carbonation of the concrete or by the presence to too much concentration of chloride ions at the steel surface and an electrochemical cell develops. For an electrochemical cell to function, an anode, cathode, electrolyte, and a conductor has to be present. For steel to corrode, the anode, cathode, and conductor are all given by the steel, the electrolyte is provided by the moist concrete [35]. One of the main purposes of concrete is to encase and protect the reinforcing steel, steel columns, and prestressing strands within it from corroding. The products of the corrosion of steel can occupy up to 7 times the volume of the original steel. This obviously causes extreme internal stresses in the concrete resulting in cracking and spalling. Therefore, it is just as important to produce a low-permeability concrete as concrete with adequate strength, especially in the cover zone [42].

The spalling of concrete in bridge decks is a serious problem, and the principal cause of this spalling is reinforcing steel corrosion. Ample cover over the steel and use of a low-permeability, air-entrained concrete will assure good durability in the great majority of cases. Additional protection, such as epoxy-coated reinforcing steel, cathodic protection, chemical corrosion inhibitors, or other means, is needed for very severe exposures. In Alabama, severe exposures only occur for bridges which span salt water or brackish water in regions along the Gulf coast.

Poor Aggregates. Quality and durable concrete cannot be made with nonquality aggregate. It is very important to identify aggregates that weather rapidly or that tend to disintegrate in solutions that will be present in their environment in concrete bridges. These unsound aggregates must be absolutely avoided to achieve quality and durable concrete.

Other primary factors causing concrete deterioration are:

Operation & Maintenance Deficiencies. The most common deficiencies in this area appear to be:

1. Not keeping expansion joints clean and operating properly

2. Not keeping all joints sealed from water and foreign matter

3. Not keeping weep holes and scuppers open and operating properly

4. Not keeping water debris cleared of substructure and river channel stabilizing system and scour prevention system in place

5. Not acting on deteriorations or damages quickly in order to minimize their effects.

Loading. Forces producing tensile stresses and/or high shear stresses (and associated inclined tensile stresses) cause concrete members to crack. The number and width of these structural cracks must be controlled by good design practices. In cases of high ADT or ADTT, due consideration should be given to fatigue damage of the affected concrete members in regions of high tensile and shear stresses and appropriate fatigue design provisions made.

Poor Concrete Mix Design. Determining the proper concrete mix design for a given application is a difficult task. For example, numerous aggregate properties have an impact on concrete durability such as:

- Gradation
- Absorption
- Soundness
- L.A. Abrasion
- Porosity
- Coatings
- Hardness
- Abrasion resistance
- Reactivity
- Thermal properties
- Resistance to freeze-thaw

Also, the appropriate type of portland cement must be selected, i.e.,

Type I - Normal
Type II - Moderate Sulfate Resistant
Type III - High Early Strength
Type IV - Low Heat of Hydration
Type V - High Sulfate Resistant
Type K - Shrinkage Compensating

the best mineral admixtures (fly ash, silica fume or both), and the appropriate chemical admixtures, such as water reducers or superplasticizers, must be selected. Obviously, many factors are involved and many combinations are possible, and this makes concrete mix design difficult. However, the importance of this task can not be overstated as the quality and durability of a concrete bridge will, to a large degree, be established by the mix design selection.

As indicated earlier, weathering causes deterioration of concrete. Hence, it is important to develop a relatively watertight concrete that has good weather-resistant characteristics. The variables that make concrete watertight are [21]:

- Richness of mix
- Gradation of aggregates
- Compaction of Concrete in place
- Curing.

One can reduce permeability of concrete by decreasing the water/cement ratio. However, this increases the curing time and causes more heat of hydration. A renewed interest in ground granulated blast furnace slag has recently become popular because it reduces permeability with a lower heat of hydration.

To help achieve durable and low permeability concrete, one should apply the four C's:

- Constituents
- Compaction
- Curing
- Cover.

Also, many good water sealer products are available and these products should be used to enhance concrete water resistance and thus enhance its durability. Concrete quality and thickness of cover must be such that chlorides, oxygen and water are prevented from reaching the reinforcing steel in concrete bridges. Takewaka and Mastumoto [57] presented the results of their investigation on this topic in the form of estimated concrete quality/cover thickness performance curves to prevent corrosion of reinforcement for a period of 30 years as shown in Figure 2.15. In this figure O.P.C. indicates ordinary portland cement and B.F.C. indicates portland blast-furnace stag cement. This figure indicates that concrete cover thickness requirements are very sensitive to concrete W/C ratio in the splash and submerged/tidal zones in marine environments.

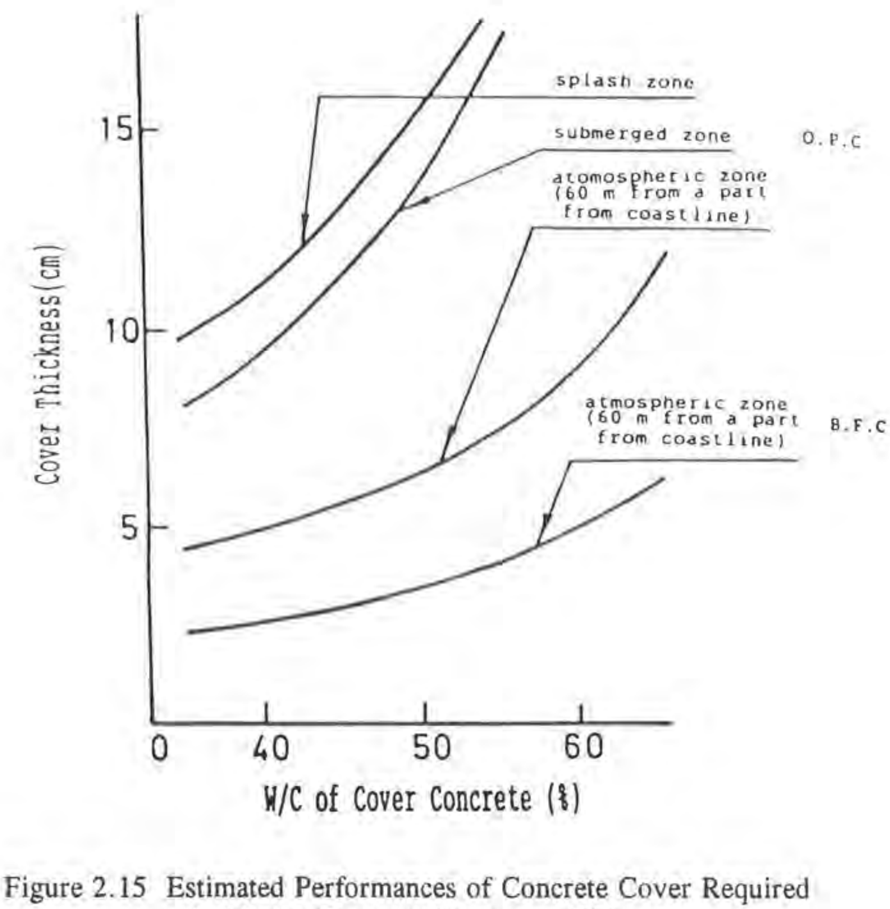


Figure 2.15 Estimated Performances of Concrete Cover Required in Marine Concrete Structures [57].

An ACI Committee addressing Concrete Durability reported that [6]

1. Good design
2. Good materials
3. Good quality control and workmanship

are all essential to the production of durable concrete. They further reported from their collective experience that ample cover over the steel and use of a low permeability, air-entrained concrete and quality deck finishing will help assure durable bridge decks. This same committee identified some of the primary factors that increased and decreased the deterioration of concrete as follows.

Factors increasing deterioration	*Factors decreasing deterioration*
Higher temperatures	Lower water-cement ratios
Increased adjacent water flow velocities	Proper cement type (in some circumstances)
Poor consolidation of concrete	Lower absorption
Poor curing of concrete	Lower permeability
Alternate wetting and drying	
Corrosion of reinforcing steel	

This same ACI committee recommended that "Protection against sulfate attack is obtained by using dense, high quality concrete with low water-cement ratio, and a portland cement having the needed sulfate resistance. Air entrainment is of benefit insofar as it reduces the water-cement ratio." It should be noted that air entrainment is essential to resist freeze-thaw exposures and is indirectly good for resistance to sulfate attack via helping to reduce the water-cement ratio and thus achieve a more impermeable concrete. Hence, quality and durable concrete for bridges in Alabama should contain approximately 5-6% air entrainment.

2.7.2. Steel

The primary deteriorations and long-term failures associated with structural steel are:

- Brittle fracture (more of a cold climate problem)
- Fatigue
- Corrosion

Fracture mechanics literature teaches us that there are three primary factors which control the susceptibility of a structure to brittle fracture, i.e.,

1. Material toughness. This material characteristic is a function of the type of stress, the state of stress (plane stress, plane strain, etc.) the temperature, and the loading rate (slow or dynamic/impact).
2. Crack size. Brittle fracture occurs when a fatigue crack initiates and then grows to a critical size.
3. Stress level. Tensile stresses (nominal, residual or both) are necessary for brittle fracture to occur.

Other factors such as temperature, loading rate, stress concentrations, residual stresses, type of stress, etc. affect these primary factors, and in turn affect brittle fracture. The life span of a structural component is determined by the time required to initiate a crack and then have it propagate to the critical size. Hence, the life of a component can be prolonged by extending the crack-initiation time, the crack propagation time and fracture toughness of the component material.

Precautions such as reaming bolt holes, smoothing (by grinding) welds, avoiding multiple pass welds were possible, requiring certified welders and quality fabrication practices, close inspection, etc. help to extend crack-initiation times in structural components. However, even after observing these precautions, minute material dislocations, welds, "nicks" are going to exist and provide locations for crack initiation and propagation. Hence, it is essential that bridge elements which undergo cyclic tension stresses be designed to hold crack propagations rates to acceptable low values and to use material with good fracture toughness.

Regarding the fracture toughness of steel, it is known to be quite sensitive to temperatures and exhibits large decreases in fracture toughness at low temperatures. However, Alabama winters are sufficiently short and mild, that even with wind chill factors, the temperatures probably do not drop low enough to significantly reduce fracture toughness of steel bridge members and components. Also, because of vehicle loadings on bridges being transmitted through relatively soft/flexible rubber tires, the impact or loading rate is sufficiently slow so as not to adversely affect the material fracture toughness.

Thus, implementing steel bridge designs which use steel with good fracture toughness properties, and which minimize tensile and shear stress values and ranges at locations of structure geometry changes and joints/connections should be design goals to achieve good bridge durability. All connections should be given careful considerations from a fatigue/fracture mechanics perspective and from theoretical design considerations as well as fabrication considerations. Additionally, all joints and other fatigue crack prone regions (high tensile stress regions) should be adequately protected from environmental and/or salt corrosion to prevent corrosion pitting and hence a source of fatigue crack initiation.

It should be noted that residual stresses in steel bridges have a potential for being substantial. The primary sources of these residual stresses are:

1. Rolling of the member (WF or plates)

2. Shop welding and fabrication

3. Field erection and welding.

These stresses do not, in general, affect the static load carrying capabilities of a bridge member; however, they do significantly affect the fatigue properties of the member and hence the bridges' durability/longevity. Good production/fabrication/construction/inspection practices should be exercised during the three stages indicated above in order that bridge residual stresses will be minimized. This, in turn, will enhance fatigue life and bridge durability.

A listing of some of the primary factors affecting fatigue performance and thus durability at various stages in the planning, design, construction and maintenance of bridges is as follows [15].

1. Material Selection
   - Mechanical properties
   - Surface finish
   - Residual stresses
   - Grain size

2. Design
   - Geometrical discontinuities
   - Type and magnitude of repetitive loads and resulting stresses
   - Rate of loading
   - Maximum stress
   - Stress ratio R
   - Size of member
   - Stress concentrations

3. Fabrication
   - Welding techniques
   - Shop practices

4. Erection
   - This phase should not introduce any new items during the erection process. The same care and attention required in fabrication is also required here.

5. Operation
   - The use of the structure in extreme of temperature should be considered in the selection of the material.
   - The operation of the structure, such as a bridge, equipment, or rolling stock, should be considered in extremes of hot and cold temperatures.

The literature reminds us that fatigue prone locations in steel bridges are at locations of

1. High nominal tensile stress
2. High nominal shear stress
3. Tensile or shear stress concentrations (at material or member geometry discontinuities), i.e.
   - At member cross-section changes or changes in member direction.
   - At cut-outs, cope locations, holes or other such changes in member geometry.
   - At connections and support points where concentrated loads are transferred.
4. Biaxial or triaxial states of stress.
5. In-plane member splices and/or joints and connections.
6. Out-of-plane diaphragm and cross-bracing connection points.
7. At all stiffners (and other attachments) on floor beams, stringers or girders in the tension regions of these members.

Regarding brittle fracture or fatigue of bridge diaphragms, it is obviously not desirable that this occur. However, it must be remembered that we are probably talking about an abrupt failure of a <u>secondary</u> member. This should not lead to catastrophic results.

A fatigue-endurance limit approach to bridge fatigue design utilizes the following [34]

$$S_e = k_{su}k_{si}k_lk_tS_e'$$

where:

S_e = the endurance limit of the part

S_e' = the endurance limit of the rotating beam specimen of the same material as the part

k_{su} = the surface factor ≤ 1

k_{si} = the size factor ≤ 1

k_l = the load factor ≤ 1

k_t = the temperature factor ≤ 1

Each of the above k factors (which are all less than or equal to one) are briefly discussed below.

1. Surface finish, k_{su}. In general, smoother surface finishes of members result in members more resistant to fatigue loading. For a smoothly polished surface, k_{su} = 1. For members that are ground, machined or cold-drawn, hot-rolled, or forged, surface finish factor values are given in Figure 2.16. The mechanical surface treatment called shot peening is often used to improve the surface finish. Shot peening involves bombarding the surface with high-velocity iron or steel shot discharged from a rotating wheel or pneumatic nozzle. This hammering effect causes the outer layer to be placed in residual compression. This residual compression works to the advantage of the material as the material is subjected to cycles of fatigue loading. If specific data are not available, it is usually considered appropriate to use a surface finish factor of 1 when a part has been shot peened.

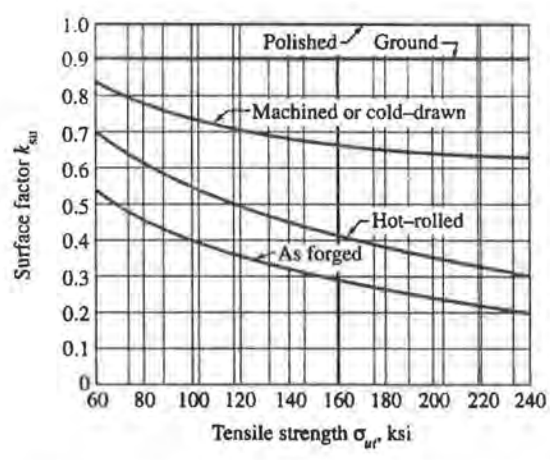


Figure 2.16 Surface Effect Influence on Surface Factor [34].

2. Size effect, k_{si}. In general, there is a greater likelihood that members with larger cross-sections will contain flaws. From experimental testing, the following empirical relationships were established for the size factor, k_{si}.

   a. Round bars in reverse bending and torsion.

      $k_{si} = 1$ for $d \leq 0.3$ in

      $k_{si} = 0.869d^{-0.097}$ for $0.3 \text{ in} \leq d \leq 10$ in.

   b. Square bars in reverse bending and tension.

      $k_{si} =$ use same expressions as in (a) but using $d = d_{eff}$ (where d_{eff} is the diameter of a circular bar of the same cross-sectional area as the square bar)

   c. Axially loaded bars.

      $k_{si} = 0.71$ if mechanical properties of material are known from actual testing

      $k_{si} = 0.60$ if no test results are available

3. Load factor, k_e. The load factor k_e for different types of loadings (and thus stresses) are given by the following equations

   $k_l = 0.9$ for axial load, for $\sigma_{ut} \leq 220$ ksi

   $k_l = 1$ for axial load, for $\sigma_{ut} > 220$ ksi

   $k_l = 1$ for bending

   $k_l = 0.58$ for torsion

Note that the endurance limit in reversed torsion was determined to be about 58% of the endurance limit in reversed bending. This result has also been predicted by the distortion energy theory of material failure. Therefore, the load factor of 0.58 is used for reversed torsion situations. It should be noted here that at the diaphragm to girder interface in bridges, the stress conditions are quite complex. Out of plane bending and shear stresses are developed as are torsional stresses due to relative movements of the bridge girders. These transverse and torsional shear stresses at diaphragm to girder interface/welds are probably a significant cause of the fatigue crack problems that occur at these locations.

4. Temperature factor, k_t. The temperature affects the endurance limit of a member. For low temperatures, brittle fracture is a possibility and should be considered before fatigue. For steels with temperatures up to around 450°C (840°F) a factor of $k_t = 1$ is typically used.

Guidelines extracted mostly from Reference 13 on designing welded steel structures for enhanced fatigue life and thus enhanced durability/longevity are as follows.

1. Always work with actual calculated stresses rather than mean stresses.
2. Reduce stress variations or ranges in fatigue prone regions of members or components while not increasing maximum or mean stresses where possible.
3. Avoid abrupt changes in member section geometry at locations of high tensile or shear stress.
4. Use smooth transition to section-geometry changes and locate these in regions of low tensile or shear stresses wherever possible.
5. Avoid sharp corners, attachments, opening, etc. at locations of high tensile or shear stress.
6. Avoid designs with fracture critical members by having some redundancy.
7. Use simple butt welds in lieu of lap or T fillet welds.
8. Avoid specifying grinding, if possible, since it is labor intensive and costly. (Grinding the reinforcement off of butt welds will increase the fatigue strength to approximately that of an unwelded plate.)
9. Avoid excessive reinforcement, undercut, overlap, lack of penetration, and roughness of weld.

10. Avoid placing welds in regions of large (relatively speaking) displacements and movements.

11. Avoid using small weld sizes on thick plates.

12. Avoid using large weld sizes on thin plates.

13. Avoid using weld sizes requiring multiple weld passes where possible.

14. Avoid eccentric load applications which cause component flexing where possible.

15. Avoid allowing large member deformations and deflections, i.e., avoid designing highly flexible welded steel bridges.

16. Avoid restrained internal sections where possible.

17. Avoid biaxial and triaxial stress conditions where possible.

18. Avoid using stiffners and cross-bracing where possible.

19. Where required, avoid having stiffness or cross-bracing welded to only one side of girder webs and flanges.

20. For all major or permanent steel bridges, design for 10^8 cycles of load application.

21. For all major or permanent steel bridges, use a high fracture toughness steel with good machinability and weldability.

22. For all major or permanent steel bridges, weathering steel should be considered the standard choice because of its corrosion resistant and good environmental related durability characteristics. Deviations from using this steel should be justifiable from a enhance durability/longevity viewpoint.

23. Use only AASHTO Fatigue Classification Categories A, B or C for members and weld and bolt connections. Use Category D, E and F welded connections only when it is not possible to use B or C.

Weathering steel has been in use in highway bridges since 1964, with over 2,000 of such structures being erected all across the country to date [30]. Experience gained from operating these bridges indicate certain problem areas causing excessive corrosion and suggestions for improvement of design and maintenance practices. Under design practices these include:

1. Expansion Joints. Expansion joints are a major cause of problems in all types of structures. Leaking expansion joints pose a particular hazard for unpainted weathering steel structures, since they permit chloride-laden runoff from deicing chemicals direct access to the weathering steel superstructure elements. In addition to deicing chemicals, dirt, sand, and other debris pass through leaking expansion joints. These material accumulate on the flanges of the superstructure members as well as on diaphragms and around the bearings. Combined with moisture coming through the joints, the debris forms an atmosphere conducive to corrosion.

Building jointless bridges, partially painting steel members, and simplifying expansion joint details were three measures in [30] identified for consideration in reducing the corrosion problem at expansion joints.

2. Scuppers. Many scuppers have been detailed on bridge plans with little regard to their design or effectiveness. Gutter scupper requirement nomographs indicate that in some circumstances, scupper spacing may be increased to as much as 1,400 ft.
3. Design Details. Avoid water- and debris-trapping details such as the one shown in Figure 2.17.

Figure 2.17 Water Trapping Stiffners.

Under maintenance practices these include:

4. Water Flushing. Periodic flushing out of expansion joints and scuppers helps to ensure that they operate as intended and helps to keep water away from steel members and bearings. Any accumulated debris, whether caused by birds, leaky joints or scuppers, or high water-that accumulates on weathering steel surfaces-should be flushed off.

5. Cleaning and Painting of Steel. Cleaning and painting the bridge steel near expansion joints is believed, by many, to be an effective method of reducing excessive corrosion at these problem points.

Few weathering steel bridges, are entirely unpainted. Many states now demand that weathering steel members be painted on either side of a joint and where water is apt to collect. Similar stipulations may appear in a new set of guidelines that FHWA is formulating, based on the experience of users as well as academic research projects [50]. There are places in the U.S. where the climate is right for weathering steel, but others where it is not. Gary Kasza of FHWA's Portland, Oregon office has seen weathering steel bridges in both very wet and very dry conditions. Medium is best, he says, citing a good example in a part of Idaho where annual rainfall averages 30 in. "This really qualified as dry, compared to southeast Alaska, where Prince of Wales Island gets more than 160 in. The Idaho bridge developed the nice, tight patina it was supposed to."

When New York Thruway officials sought to replace the collapsed Schoharie Creek Bridge in April 1987, one of the criteria they set forth, was use of weathering steel so that the bridge would not need painting. The others were redundancy and speedy construction [50]. They decided the best way to design the bridge and get it built quickly was to shorten the steel fabrication time, and the best way to do that was to eliminate as many details as possible. To eliminate stiffeners, they thickened the web. They also kept the flange widths constant for the entire 720 ft of the bridge, and limited the number of flange splices. Thickening the webs not only increases their corrosion resistance, but that the heavier members help stiffener the bridge and reduce differential deflection under live loads. For redundance, the new bridge has 14 stringers instead of the original two girders.

Significant opinions about the use of weathering steel were gathered by FHWA in a recent forum on this topic. These opinions are summarized in the guidelines below.

- Avoid extreme environments that interfere with the wet/dry cycles necessary to form the tight, protective oxide. Avoid atmospheres containing salts or corrosive industrial fumes; tunnel-type locations where salty road sprays are sprayed on the superstructure; applications where the steel is constantly submerged in water or soil.
- Avoid retention of water and debris, such as horizontal members, joint angles that form "cups," or drainage paths that are easily stopped up. Pay attention to draining the roadway below.
- Protect the steel below deck joints. Troughs and downspouts must be kept open and carry water well away from the steel. Use sealed deck joints, but remember that most will leak when subjected to traffic and bridge movement. Protect steel beneath joints by painting 5 or 10 ft on either side of the joint with an inorganic zinc-rich coating in a color to match the unpainted weathered steel.
- Consider jointless bridges such as those designed in Tennessee that use integral abutments and a cap supported by H-piling that absorbs girder movement.
- Do not combine an asphaltic deck/steel flooring system with weathering steel superstructure. Alternates are reinforced concrete decks 8-10 in. thick and grid

decks, open or filled with concrete.

- Protect the substructure against staining by temporarily wrapping piers and abutments during construction, and then directing water away from them with proper attention to joints, troughs, drainage systems and drip plates. Various protective coatings are available for the concrete.
- If necessary, consider galvanizing the A588 where it abuts joints. Consider painting outside girder (fascia) to relieve expansion differential due to heat absorption in direct sunlight.
- Flush the bridge regularly to wash off salts and prevent debris buildup. In some areas, local fire companies will do this during training sessions.
- Keep weeds away from weathering steel to promote air circulation and improve the overall bridge appearance.

2.7.3 Prestressed Concrete

Prestressed concrete is a relatively new bridge material that has steadily gained popularity since its introduction in the late 1950's. It has some similar characteristics to that of concrete, e.g., they are both subjected to concrete cracking and their reinforcement will corrode. Although some cracking of the reinforced concrete is expected, prestressed concrete should not crack since the concrete is in compression due to the prestressing tendons [22]. If the prestressing tendons of a prestressed concrete bridge were allowed to corrode, there would be a substantial loss of strength.

The prestressed fibers can be corroded by [35]:

- General corrosion
- Localized corrosion

- Stress corrosion: This type corrosion is the cracking of an alloy that results from both corrosion and static tensile stresses.
- Hydrogen embrittlement: In this type of corrosion, the ductility of the steel is reduced as the hydrogen content is increased. This can cause brittle behavior and sudden failure.
- Galvanic Corrosion

Stress corrosion and hydrogen embrittlement are more likely to occur in prestressed concrete because of the highly stressed steel used in prestressing.

A research study by the U.S. Army Corps of Engineers found that epoxy-concrete end caps provided the best protection to the tendon end anchorages. Also, they found that the post-tensioning wires were not damaged wherever they were encased in a metal duct and protected with a portland cement grout [35]. Other studies have shown a loss of 30-40 percent of strength of prestressed concrete girders over a twenty year period. One study indicated that some twenty-five year old prestressed concrete highway bridges in Italy showed a loss of over 60 percent in prestressing tendons due to corrosion.

A major concern when using prestressed concrete is the difficulties of inspection and repair. Inspection is very important, especially around the anchorage zones. External tendons have been used in highway bridges, where tendons can be inspected. It is also important to repair leaking joints and drains, and to remove accumulated deicing salt/sand. This will help keep chlorides out of the system, and damage to a minimum.

There have been few reports of failures of prestressed concrete. Those failures that have been reported can be linked to design, material, or construction flaws.

Many design procedures that have already been mentioned also apply to prestressed concrete. For example, the use of quality, low-permeability concrete is very important to good prestressed concrete design. Also, adequate cover of concrete over the prestressing tendons is needed. Continuous decks generally require post-tensioning tendons located in the deck slab. The Pennsylvania Department of Transportation requires that the longitudinal post-tensioning ducts should be at least 10 inches below the top of the deck slab. Also, corrosion protection is needed for the ordinary reinforcement of prestressed concrete members just as it is needed for reinforced concrete members.

In addition to the regular design for concrete, there are some provisions that need to be accounted for in the design of prestressed concrete. Provisions should be provided to have effective drip checks. Also, provisions could be made for the installation of additional tendons after the structure has been placed in service. This would include anchor blocks, tendon saddles, and holes in diagrams for tendons. Also one can design to minimize early thermal shrinkage cracking by supplying many small diameter bars at close spacings [35].

Grouting is also something that needs to be considered in the design. Inspections have revealed that ungrouted lengths of tendons near the road surfacing are a special corrosion problem for post-tensioned decks. It has been recommended that extra local deck waterproofing be added to areas that have the post-tensioning near the deck surface. Pre-tensioning does not rely on grouting for strength. It also takes advantage of the tendons being in the lower half of the beam which allows for more concrete cover [42].

Designs that are easy to construct and that recognize the site conditions and the sophistication of the contractor increases the likelihood of a durable bridge. For example, tendons that have the minimum possible change in directions and that detail for duct venting are good designs. Also, on complex structures, it is wise for the designer to provide for grouting trails to verify results by postmortem examinations as well as specify for quality-assurance measures for the entire construction stage.

Prestressing tendon ducts should have the following properties:

- Will not allow paste or mortar to enter
- Sufficient strength to prevent damage while installing
- Will not allow the tendons to cut or crush the walls
- Chemically stable
- Transfers bond between grout and surrounding concrete.

During construction, it is good practice to keep the storage time of the prestressing wires to a minimum. If they are stored, they should be covered with a rust-preventive oil. Also, it is important to store it in dry conditions and avoid direct contact with the ground. One needs to keep the diameters of the coils large enough to prevent a permanent set in the tendons. Grouting soon after the prestressing fibers are in place in post-tensional systems is very important. However, sufficient time needs to be given to allow stresses to equalize and stabilize prior to grouting [35].

As indicated earlier, a major concern with prestressed concrete bridges is in the area of repair and rehabilitation. The paragraphs below describe the rehabilitation of a prestressed concrete box beam deck of the F.G. Gardiner Expressway in Toronto, Canada to illustrate that, though more challenging, such repair and rehabilitation procedures can be successfully implemented.

The existing deck structure of the Gardiner Expressway, consisting of simply supported prestressed precast concrete box beams which support a thin concrete topping with waterproofing membrane and asphalt, exhibited serious deterioration and was scheduled for rehabilitation in 1981, 17 years after construction [58]. The major cause of deterioration was identified to be leakage of salty water from the deck through transverse expansion joints and longitudinal joints between individual beams.

Although still very limited, the rusting of prestressed strands at beams soffits was identified as the most dangerous aspect of deterioration and was repaired by latex-modified shotcrete in 1981 and 1982. In the spring of 1984, the simply supported spans were made continuous by a new reinforced concrete deck slab; nearly three fourths of the original expansion joints were eliminated. The continuous construction increased the strength of the box beams and provided a very welcome strength reserve to cover potential future strength losses through rusting of strand.

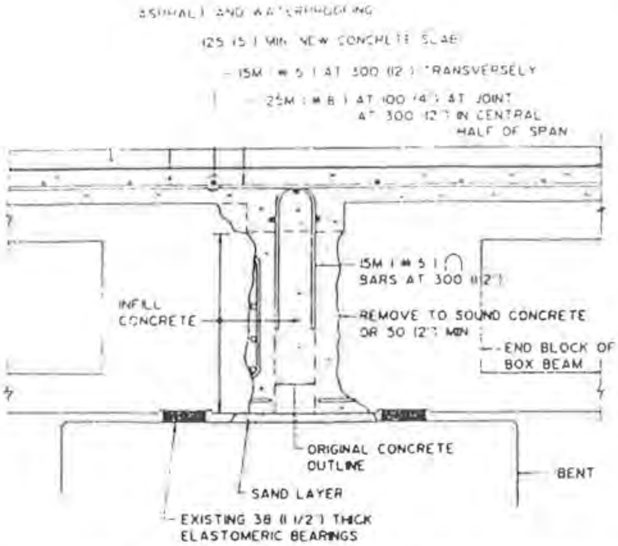
At the remaining expansion joints, modern joints with replaceable seals were installed which are expected to remain substantially watertight with regular maintenance. A new drainage system with open hoppers was installed. The new system with simple maintenance features is expected to remove the surface water away from the deck quickly and reliably, and thereby reduce the danger of salt-water leakage through the joints and deck. On top of the deck a new pliable waterproofing membrane and new asphalt pavement were installed. Figure 2.18 graphically summarizes the rehabilitation described above for the Gardiner Expressway.

2.8 Results of Bridge Deterioration Studies

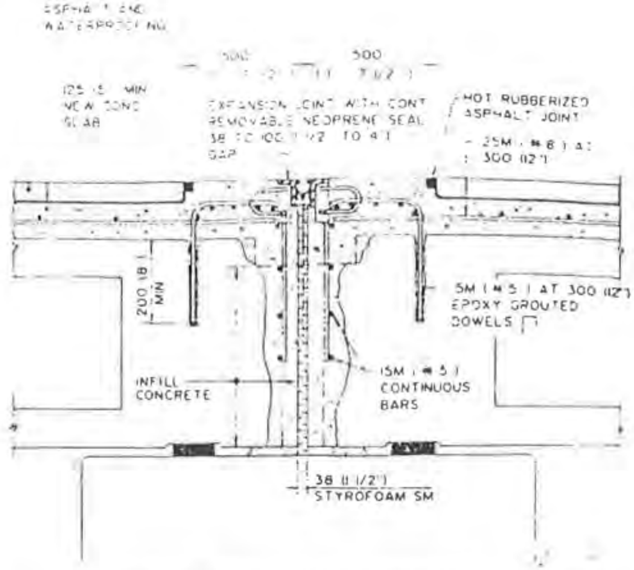
Brief summarizes of works by other researchers and/or transportation agencies in assessing bridge deteriorations are presented below. These results provide excellent insight regarding bridge durability/longevity both qualitatively and quantitatively. That is, they speak to the main factors affecting durability and quantify actual bridge design lives currently being achieved.

2.8.1 Massachusetts Transportation Systems Center (TSC) Study [60]

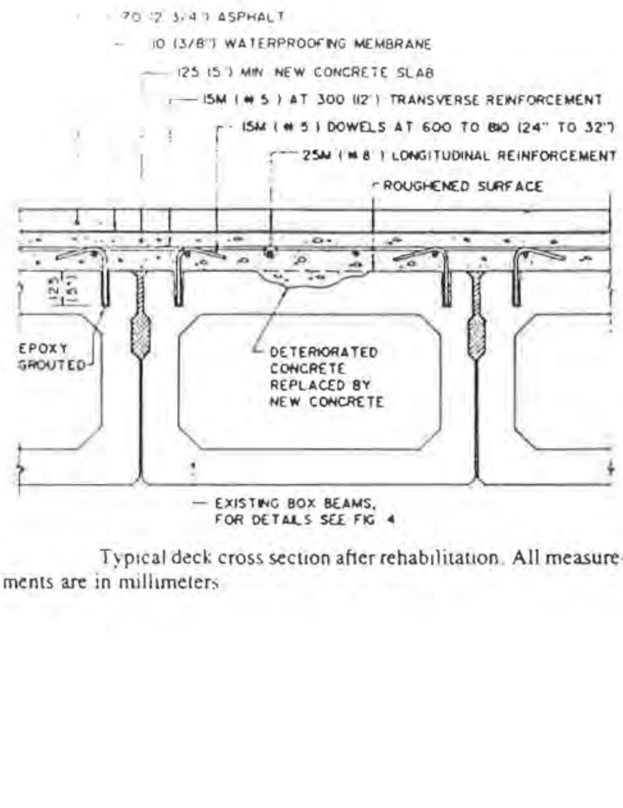
The Massachusetts TSC developed empirical deterioration models which predicted deck, superstructure and substructure condition ratings based on variations in average daily



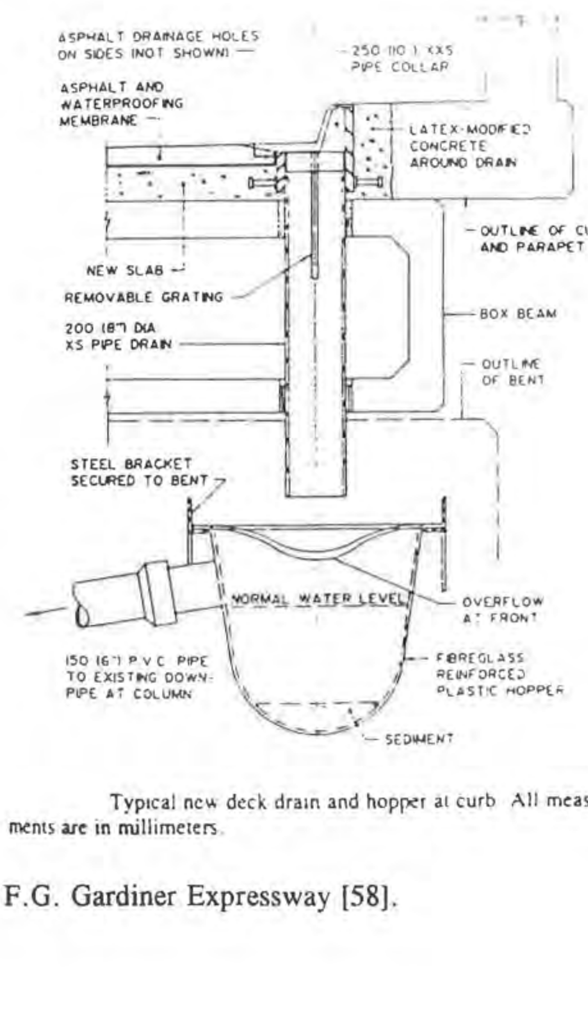
Typical continuous transverse joint after rehabilitation. All measurements are in millimeters.



Typical expansion joint after rehabilitation. All measurements are in millimeters.



Typical deck cross section after rehabilitation. All measurements are in millimeters.



Typical new deck drain and hopper at curb. All measurements are in millimeters.

Figure 2.18 Rehabilitation Details for F.G. Gardiner Expressway [58].

traffic (ADT), bridge age, main structure type, skew angle, number of spans and ownership. Results of the study indicated that all of these variables were correlated with bridge condition ratings. However, they found age and ADT were the two most influencing variables. They found that bridge decks deteriorated slightly faster than either the superstructure or substructure, and deck deterioration was more sensitive to ADT. They determined the national average deterioration rates shown in Table 2.4.

TSC used a screened data base of 151,933 bridge records from bridges all across the nation. It is interesting to note that they eliminated bridges over 25 years old in their screening. Hence, their deterioration rates are those for relatively new bridges.

2.8.2 Wisconsin DOT Study [60]

The Wisconsin DOT investigated relationships between bridge condition rating and bridge age using 4,463 Wisconsin bridges. Five different bridge types plus culverts were included in their population sample, and their piece-wise linear regression analysis results are shown in Table 2.4.

2.8.3 New York DOT Study [60]

The New York DOT used a piece-wise linear regression analysis to develop a bridge deterioration model which correlated bridge condition rating with age. They employed bridge condition data from two inspection cycles which were spaced two years apart. Using this approach they determined that the average deterioration rate for all structure components, i.e., deck, superstructure and substructure, was 0.122 points per year.

2.8.4 North Carolina DOT Study [60]

The North Carolina DOT turned from statistical procedures to other methods to construct bridge deterioration models because statistical procedures were ineffective for their

Table 2.4
Bridge Deterioration Rates
From Other Studies.

Investigating Agency	Bridge Sample	Avg. Deterioration Rate[1]			Comments
		Deck	Superstructure	Substructure	
TSC	151,933[2]	0.125	0.10	0.10	
Wisconsin DOT	4,463[3]	-	-	-	0.07 for 0-25 years 0.07 for 25-45 years 0.19 for 45-70 years
New York DOT	-[4]	0.122	0.122	0.122	
Virginia TRC	571[5]	-	-	-	No Deteriorations Rates Given
ALDOT	3,540[6] State Owned	0.076 0.225	0.082 0.178	0.078 0.182	Approx. 0-46 yrs Approx. 46-77 yrs
ALDOT	7150[7] Non-State Owned	0.093 0.320	0.096 0.447	0.085 0.695	Approx. 0-40 yrs Approx. 40-53 yrs

[1] Given in decrease in condition rating number per year.
[2] Bridges from throughout the U.S. which were less than 25 years old.
[3] Sample consisted of 6 Wisconsin bridge types.
[4] Sample was of New York bridges.
[5] Random sample representing 4 Virginia bridge types.
[6] Approximately 3,540 state owned Alabama bridges.
[7] Approximately 7,150 non-state owned Alabama bridges.

data base. From their study they did conclude the following:

- For all three major bridge elements (deck, superstructure, substructure), material type was a key variable.
- ADT was a significant predictor variable for deck deterioration, with higher-ADT bridges exhibiting higher deterioration rates.
- Structure type and bridge system (interstate, primary, secondary system) were significant predictors for superstructure deterioration. Superstructures on the interstate and primary systems had higher deterioration rates than those in the secondary system.
- Geographic location of the bridge was a predictor for substructure deterioration. Bridges in coastal areas had higher deterioration rates than bridges in the piedmont and mountain areas.
- Bridge condition deteriorates with age, and older bridges deteriorate at a faster rate.
- Substructures deteriorate slower than decks and superstructures.
- Among the three major bridge elements, decks had the highest deterioration rate.

2.8.5 Virginia Transportation Research Council (TRC) Study [60]

The Virginia TRC used a sample of 571 randomly selected bridges (50 years and younger on primary routes and 30 years and younger on secondary routes). The sample represented four bridge types - (1) steel beams with timber decks, (2) steel beams with concrete decks, (3) concrete beams with concrete decks, and (4) concrete box beam bridges. Their primary conclusions are as follows:

- Bridge type, age, number of spans, and previous joint and deck ratings proved to have the greatest influences on substructure and superstructure ratings.

- Steel beam, concrete deck and multispan structures tended to deteriorate more rapidly than concrete beam or concrete box-beam structures.
- The deterioration model for concrete decks showed that decks on multispan bridges tended to have lower ratings than decks on single-span bridges, other things being equal. Hence, the number of spans was an explanatory variable.
- The model also showed that the logarithm of bridge age and the logarithm of cumulative traffic volume had significant negative influences on concrete deck ratings. That is, higher traffic volume and greater bridge age usually corresponded to lower deck ratings. This model had a correlation coefficient (r-squared) of eighty-five percent, which meant that eight-five percent of the variation in concrete deck ratings could be explained by the corresponding variations in bridge span type (whether single-span or multi-span), bridge age and cumulative traffic volume.
- For concrete decks rated seven or below, the correlation of predictor variables with deck condition was even better. The r-squared was an impressive 93 percent.

2.8.6 Alabama Department of Transportation (ALDOT) Study [60]

The University of Alabama conducted a bridge deterioration investigation as part of their Bridge Management System (BMS) work to develop bridge deterioration models for use in ALDOT's BMS. They looked at the most current Structure Inventory and Appraisal (SI&A) data during the 1990-91 time period on all 15,500 bridges (including culverts) in Alabama at that fixed point in time, i.e., they had a "snapshot" of bridge appraisals/conditions rather than time-history information on the bridges. For a preliminary analysis they lumped all of the data together and prepared average bridge age vs. condition rating curves for the major bridge elements, i.e., decks, superstructure, substructure. Their results are shown in Figure 2.19.

Next, they stratified the data according to factors such as material type, structure type, function class (interstate, arterial, major, minor, local), bridge or culvert, ADT and ADTT, and ownership (state-owned or non-state owned). During this process, they noted that some data sets or strata had grossly dissimilar characteristics and should not be merged, but rather each looked at as a separate problem. This point is graphically illustrated by the depiction of two grossly dissimilar data sets in Figure 2.20.

After performing a comprehensive statistical analysis of the Alabama SI&A data (or the Alabama portion of the NBI data), the University of Alabama researchers concluded that statistically procedures alone could not provide appropriate bridge deterioration models. It should be noted that the North Carolina DOT had earlier came to this same conclusion for their bridge data. This portion of the study did result in identifying, in a quantitative manner, that bridges and culverts should be looked at separately (which most would agree with). Furthermore, the researchers decided to separate both bridges and culverts into state-owned and non-state-owned categories and look at each separately. It is questionable whether ownership should be the major factor or the type of material and bridges that are more strongly associated with the two ownership categories, e.g., numerous lower capacity timber bridges for non-state-ownership and higher capacity concrete and steel bridges for state ownership.

The researchers decided to plot the data from the approximately 15,500 bridges and culverts in Alabama, separated in the categories indicated in the previous paragraph, in the manner of that shown in Figure 2.21 and to develop simple bilinear graphical models for each category. Their procedure is illustrated in Figure 2.22. They used findings from their earlier regression studies to make the Age vs Condition plots and then fit bilinear curves to the data. Their results employing this approach are shown in Table 2.5 and Figure 2.23.

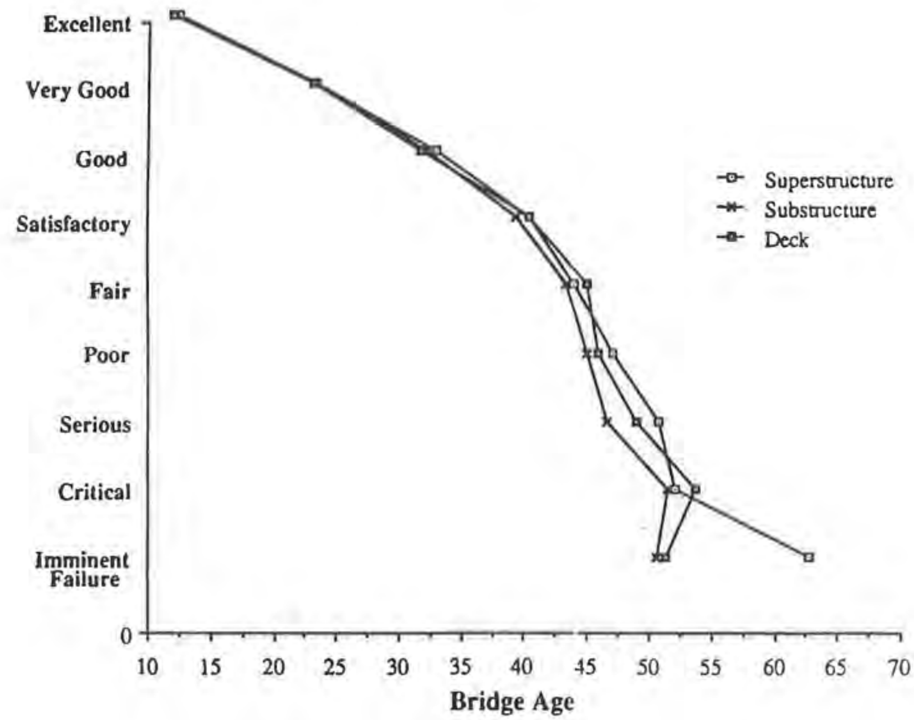


Figure 2.19 Average Ages of Alabama Bridges for Various Condition Categories [60].

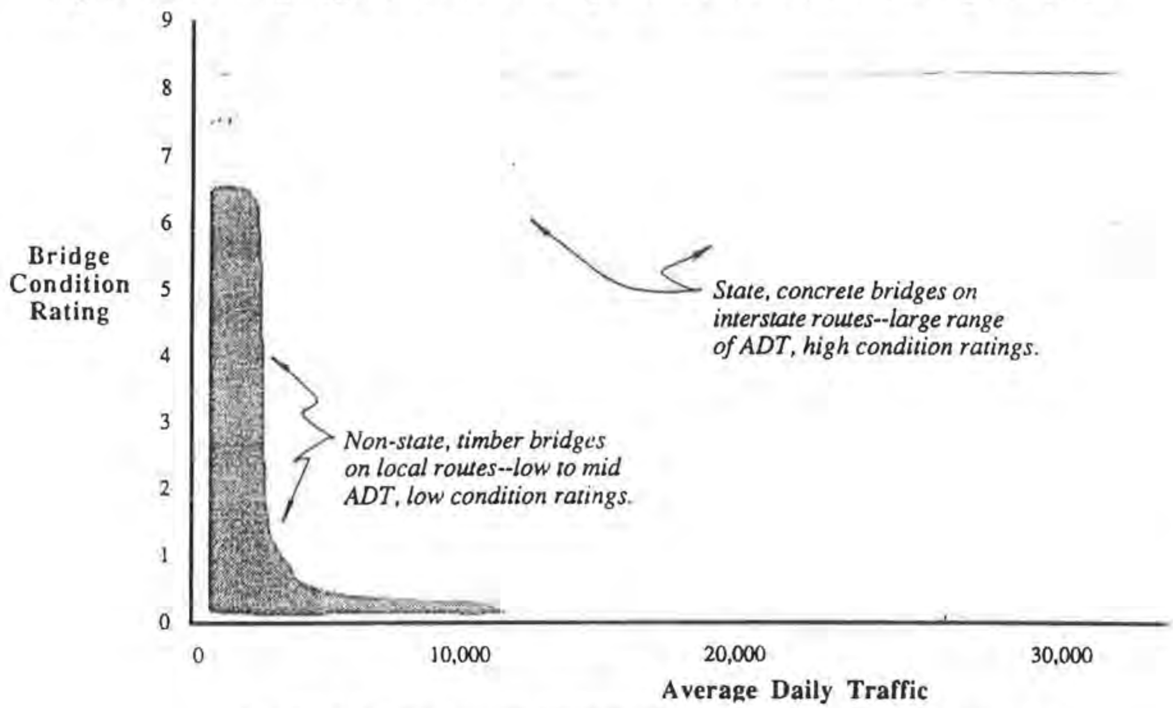


Figure 2.20 Categories of Bridges with Dissimilar Characteristics [60].

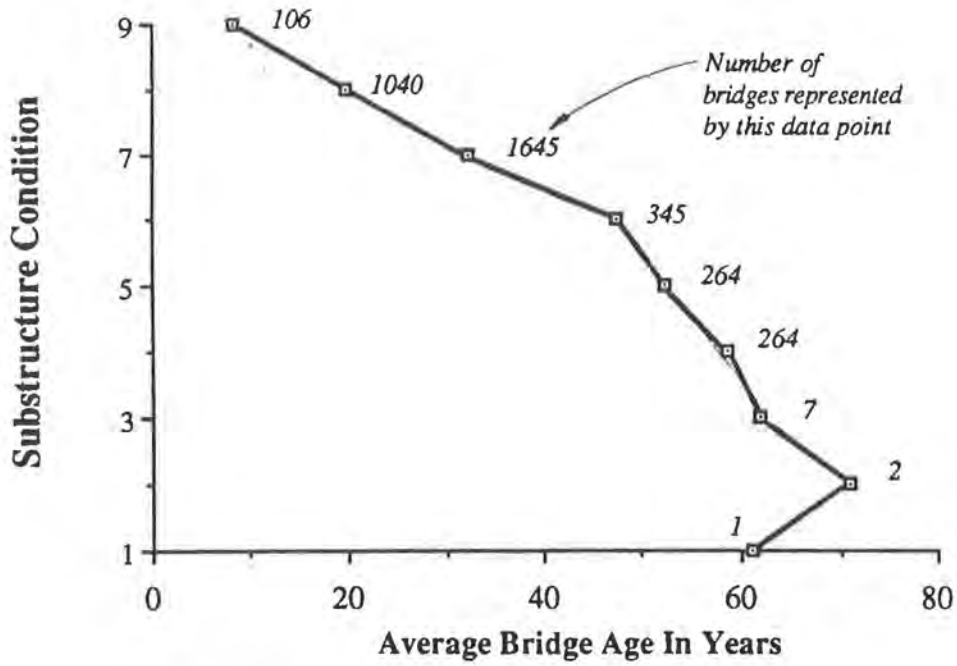


Figure 2.21 State Bridges, Substructure Deterioration [60].

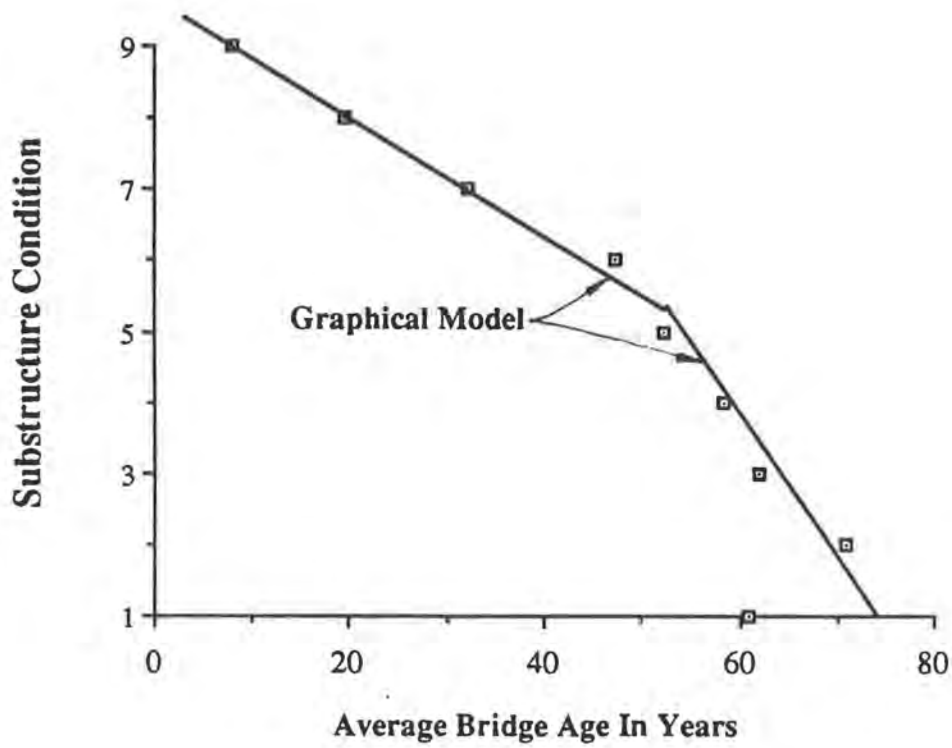


Figure 2.22 Graphical Model [60].

Table 2.5
Two-Stage (Graphical) Deterioration Models [60]

Bridge Type	Rating at Age Zero	Condition Points/Yr. (1st Slope)	Hinge Point Age	Hinge Point Rating	Condition Points/Yr. (2nd Slope)	Age at Rating Zero
State Deck	9.5	0.076	46.5	6.0	0.225	73.2
State Superstructure	9.7	0.082	45.0	6.0	0.178	78.7
State Substructure	9.7	0.078	47.5	6.0	0.182	80.5
Non-State Deck	9.8	0.093	38.0	6.3	0.320	57.7
Non-State Superstructure	9.5	0.096	42.0	5.5	0.447	54.3
Non-State Substructure	9.4	0.085	40.0	6.0	0.695	48.6

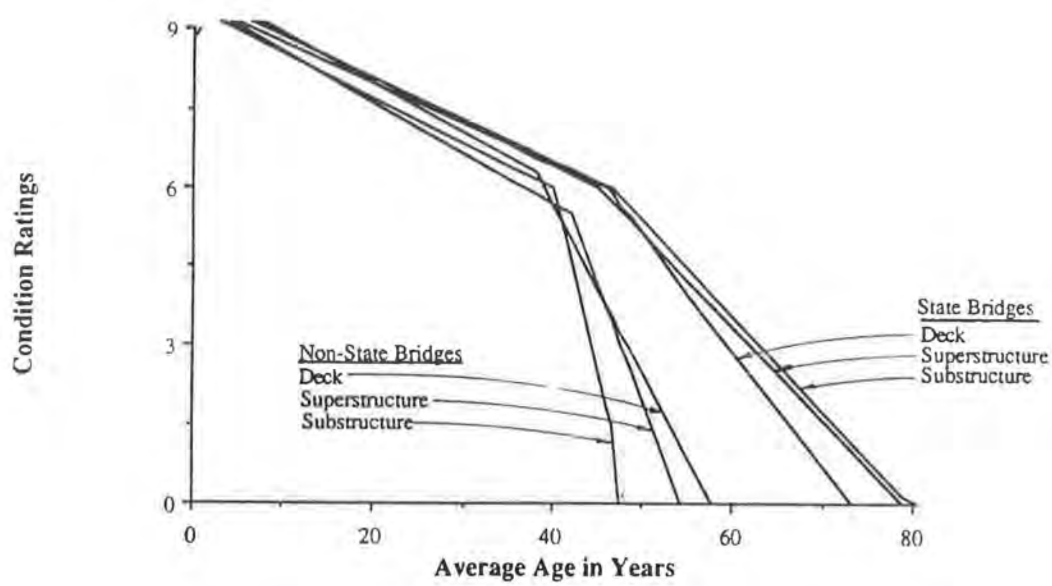


Figure 2.23 Recommended Deterioration Models [60].

2.8.7 City of New York Bridge Study [62]

The primary results of this study are presented graphically in Figures 2.24 - 2.30. The numerical ratings shown in each of these figures are based on the rating system indicated below.

Numerical Rating	Description
1	Potentially Hazardous
2	Used to shade between a rating of 1 and 3
3	Serious deterioration, or not functioning as originally designed
4	Used to shade between a rating of 3 and 5
5	Minor deterioration, and is functioning as originally designed
6	Used to shade between a rating of 5 and 7
7	New condition
8	Not applicable
9	Unknown (due to inaccessibility, e.g. footings or piles)

Note in Fig 2.25 that the deterioration rate for rehab bridges is slightly greater than for the "original" bridges as would be expected, but the difference in this rate is quite small. Also, it appears that the deterioration rate after rehab is almost independent of the age at rehab. It is interesting in Fig. 2.26 that the condition ratings drop rather dramatically for the first 50 years of bridge age and then remains about constant for the second 50 years. It is not known if bridge rehabs are playing a major part in this. Figures 2.27-2.29 indicate that bridges without joints out-perform those with joints in all cases studied. In each case, the performances were quite similar for the first 10 years, after which, the jointed and unjointed

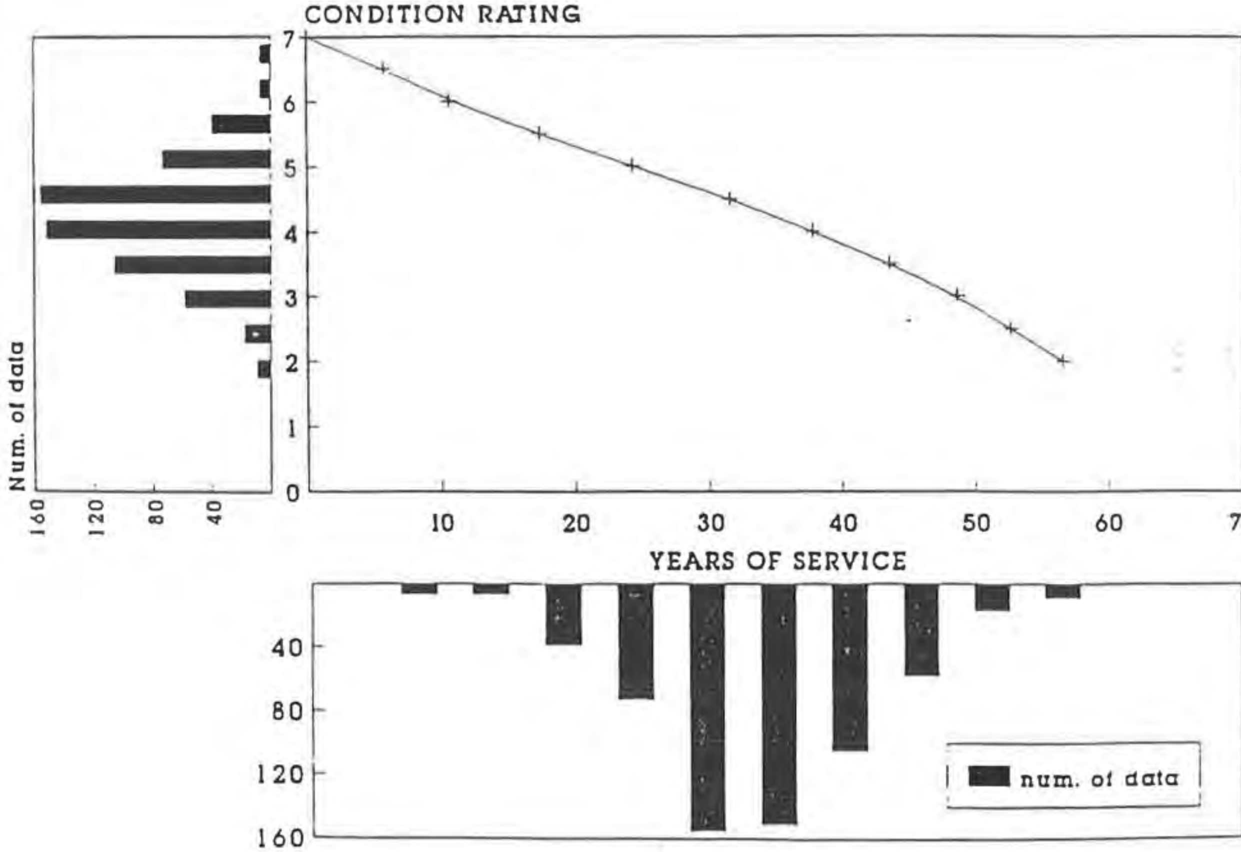


Figure 2.24 Bridge Condition Rating vs. Age and Distribution of Inspection Data from Bridges in NYC Without Major Rehab [62].

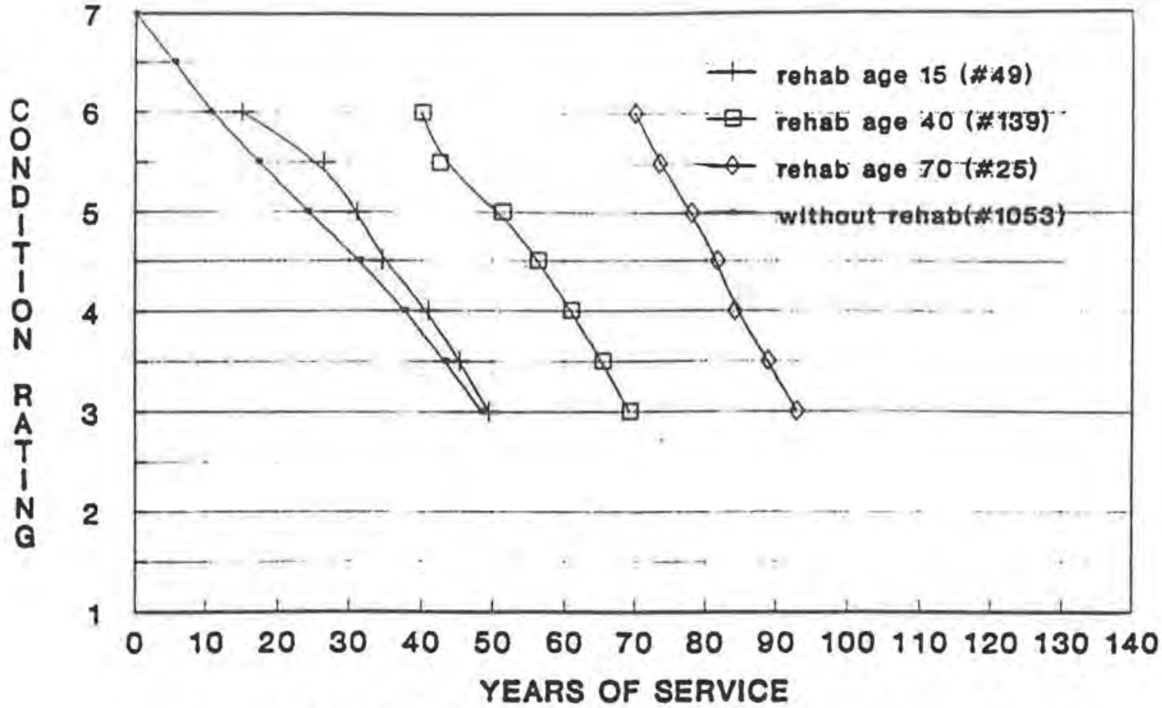


Figure 2.25 Bridge Condition Rating vs. Age for Bridges With and Without Rehab in Metropolitan NY [62].

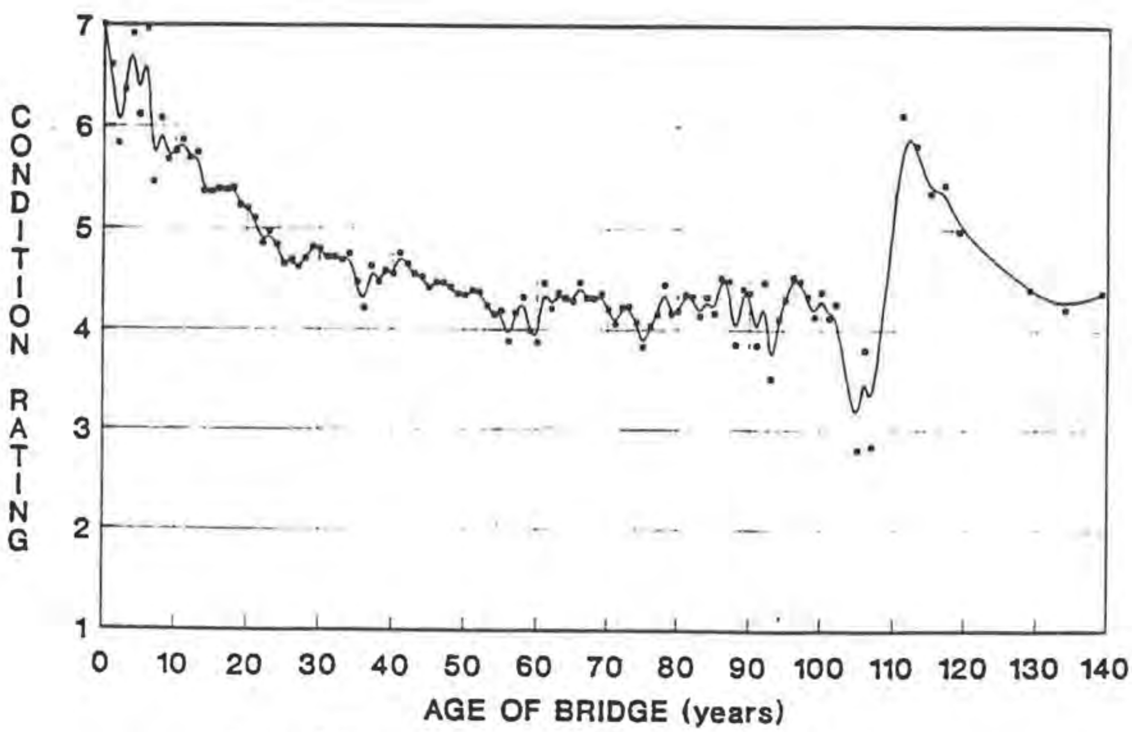


Figure 2.26 Bridge Condition Rating vs. Age for 2034 Bridges in Metropolitan NY [62].

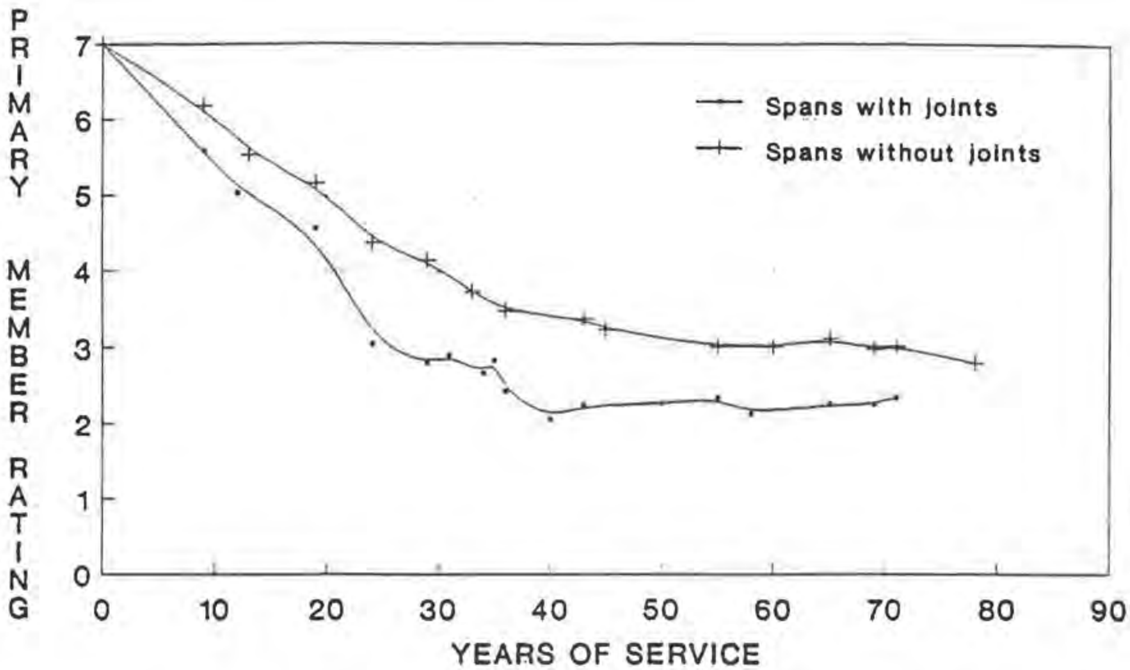


Figure 2.27 Steel Primary Member Rating vs. Age for Bridge Spans With and Without Joints [62].

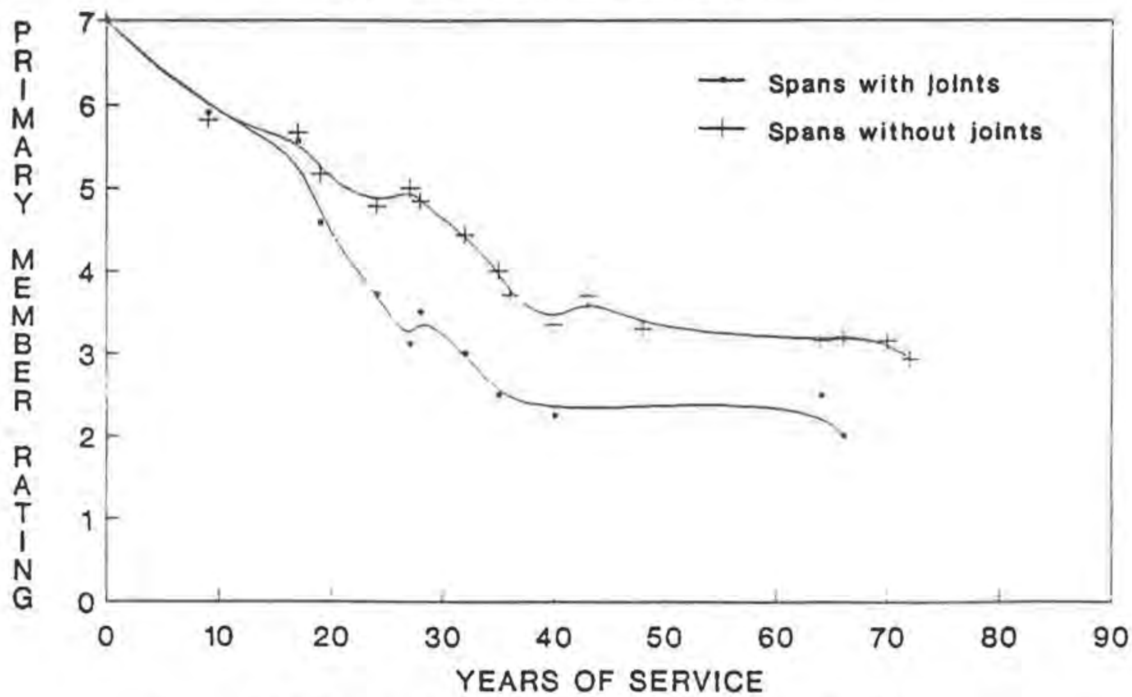


Figure 2.28 Concrete Primary Member Rating vs Age for Bridge Spans With and Without Joints [62].

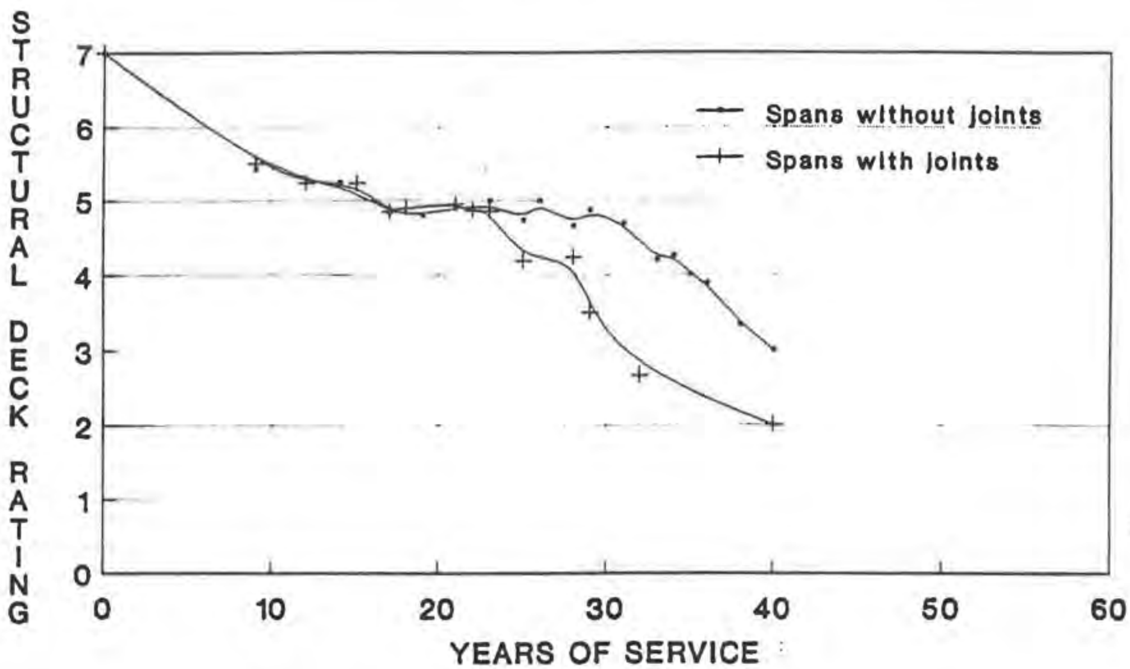


Figure 2.29 Structural Deck Rating vs. Age for Spans with Overlay [62].

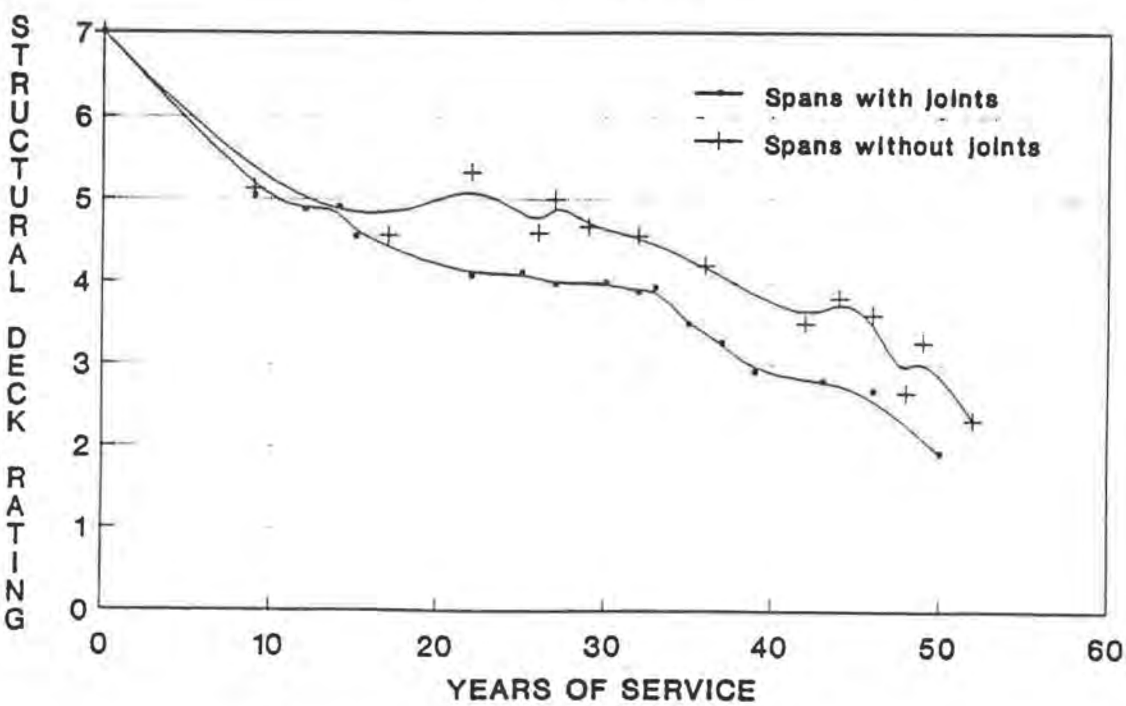


Figure 2.30 Structural Deck Rating vs. Age for Spans Without Overlay [62].

spans tracked significantly different performance curves.

2.8.8 Longevity Performance of Highway Bridge Material Study [24]

Dunker and Rabbat conducted a rather comprehensive study using the National Bridge Inventory (NBI) data for highway bridges to assess the durability/longevity performances of these bridges, and to assess how and why bridges in the U.S. highway system are failing. They used structural deficiency figures in the NBI data set as a measure of bridge durability/longevity performances. Some of their results are shown in Figs. 2.31-2.34.

Figure 2.31 shows the average percents that are structurally deficient by state and by type of superstructure material. It appeared from this figure that prestressed concrete performed better than reinforced concrete, which performed better than structural steel, which performed better than timber. For Alabama, in excess of 25 percent of the structural steel bridges were structurally deficient while under 10 percent of the reinforced concrete and under 5 percent of prestressed concrete bridges were deficient. Figure 2.32 divides the bridge types up into the different highway systems of which they are a part. Figure 2.33 shows the percent of structurally deficient bridge types verse year built. The relative performances of the four bridge materials remains as indicated above.

Figure 2.34 is a modified version of Fig. 2.33 where the bridge data is separated based on highway systems, i.e., bridges on interstate and federal routes where high and uniform maintenance standards are enforced nationwide, and those on state, county and city roads. In doing this, Figure 2.34 speaks to the importance of maintenance on bridge longevity performance. It should be noted that the solid line for timber bridges ends at 1959 because after that date very few timber bridges were built in federal routes. Also, note in Fig. 2.34 that it appears that reinforced concrete bridges and prestressed concrete bridges are less sensitive to maintenance activities than steel and timber (the dotted and solid curves are

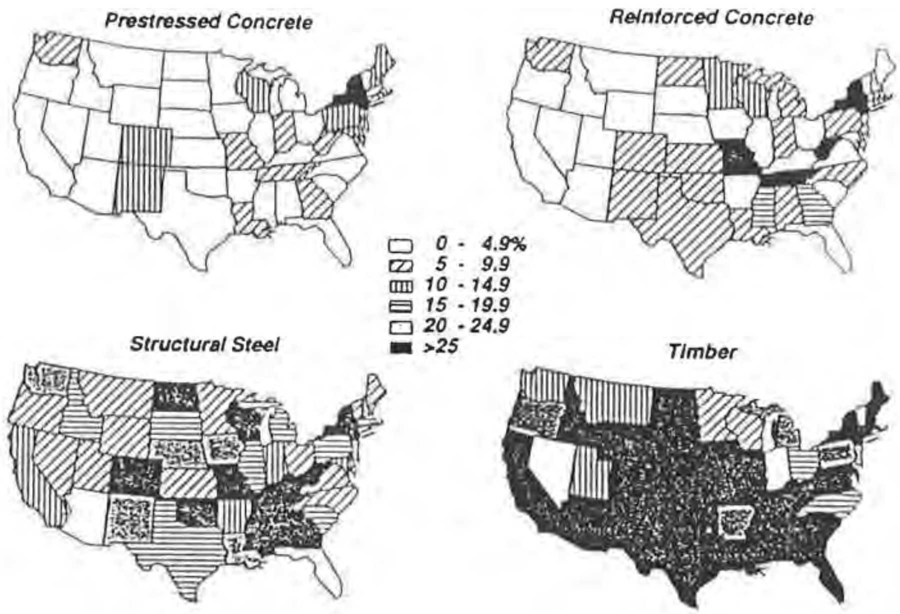


Figure 2.31 Average Percent Structural Deficiency by State for Four Bridge Types Built Between 1950 and 1988 as of May 1989 NBI [24].

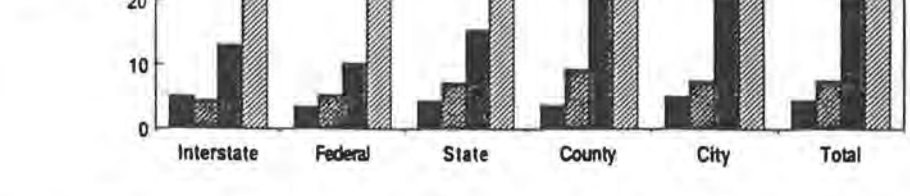


Figure 2.32 Percent Structural Deficiency by Bridge Type on Different Highway Systems for Bridges Built Between 1950 and 1988 as of May 1989 NBI [24].

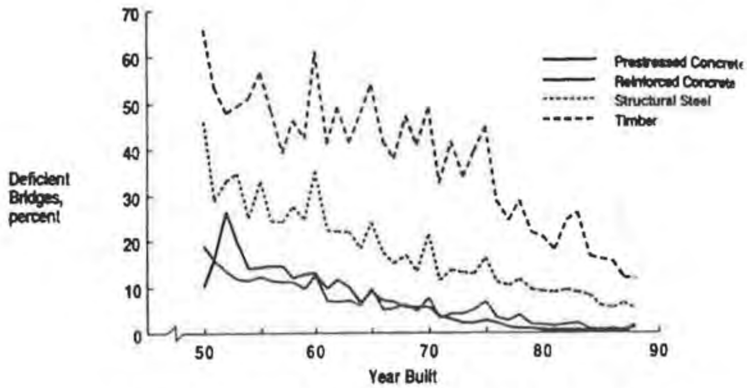


Figure 2.33 Percent of Structurally Deficient Bridge Types vs Year Built as of May 1989 NBI [24].

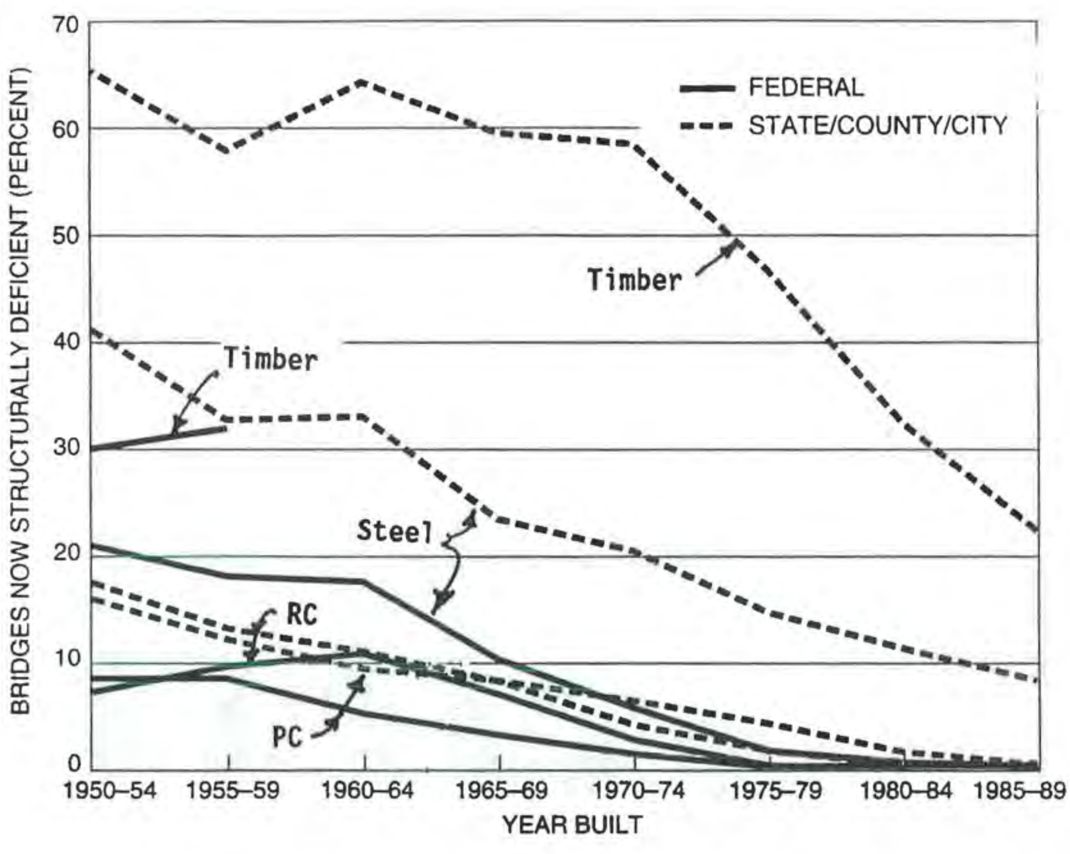


Figure 2.34 Percent of Structurally Deficient Bridge Types vs Year Build on Federal and Nonfederal Route [24].

much closer for the concrete bridges). This is probably as one would expect.

2.8.9 OECD Study [45]

Results of a 1988 survey made by the OECD of reasons for bridge closures, factors influencing service life and estimation of service life are presented in Tables 2.6 - 2.8 and Figure 2.35. In the tables the country names are provided where they are known. With the exception of Japan, it appears that the mean service lives of bridges in the OECD countries are approximately 15 percent greater (80 years vs 70 years) than in the U.S. In the case of Japan, Fig. 2.35 seems to indicate most of the bridge replacements are due to improvement works and operational problems rather than structural deficiencies.

2.8.10 Concrete Parking Garage Study [33]

Litvan and Bickley reported the results of a field survey of 215 concrete parking garages in Toronto, Ottawa and Montreal, Canada to assess the durability performances of these structures. Though they did not look at highway bridges, their data and results have applicability to concrete bridges. They concluded that durable concrete garages can be built, and that poor performances were attributed to design and construction practices. Some of their most noteworthy conclusions are as follows [33]:

- Repair by the "patch and waterproof" method lowers the rate of deterioration significantly, i.e, to approximately 30 percent of that prior to rehabilitation. Such repairs do not, however, completely stop further corrosion.
- There are no detrimental effects from installing a waterproofing membrane over concrete with elevated chloride concentration.
- Durable garages can be built without exotic methods. High-quality dense concrete, adequate cover, effective waterproofing, and good drainage seem to be essential.

Table 2.6
Reasons for Bridge Closure in Five OECD Countries [45].*

Reasons	Great Britain	Denmark	Japan	Netherlands	Sweden
Structural Inadequacy					
Design/construction	20			25	
Repair impossible	20	80		25	35
Many elements to replace			1		
Environmental actions (salt, frost)	10		63	25	42
Catastrophic failure (collapse)	10		4	5	
River upstream			5		2
Traffic on the bridge	15	20	23	10	10
Impacts under the bridge	5				1
Maintenance faults	20			10	3
Other			4		7
Total	100%	100%	100%	100%	100%
Functional Inadequacy					
New standard			36	15	68
Road/canal widening	50	25	22	75	6
Heavier traffic	10	12			6
Bottleneck: clearance	10			5	1
Bottleneck: loads	10		30	5	10
Road set out of service	20	38			7
Other		25	12**		2
Total	100%	100%	100%	100%	100%
Structural inadequacy (%)	40	27	19	10	47
Functional inadequacy (%)	60	73	81	90	53

* Values shown are percents
**Seismic retrofitting

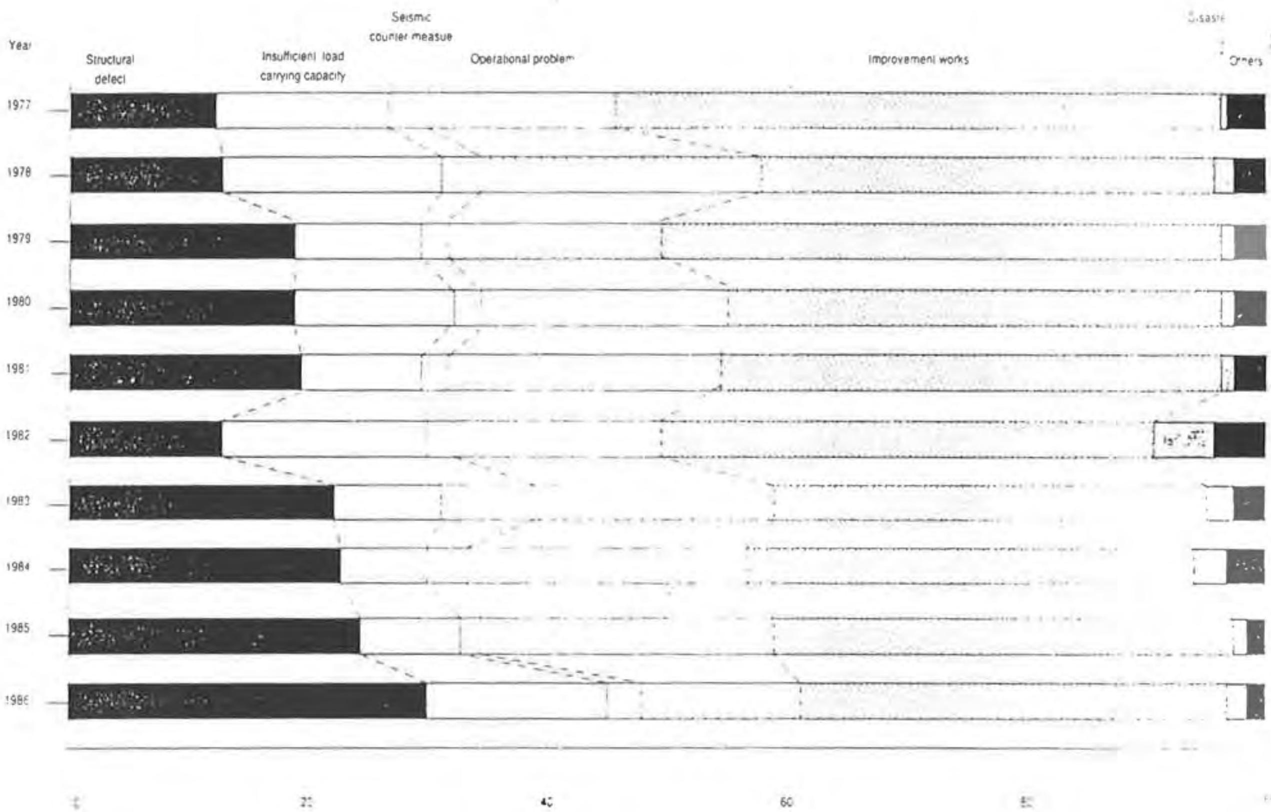


Figure 2.35 Reasons for Bridge Replacement in Japan Between 1977 and 1986 [45].

Table 2.7
Measures Influencing Increasing Bridge Service Life
for Seven OECD Countries[1] [45].

	Great Britain	CH	Denmark	Norway	Netherlands	Sweden	SF
Measures to Delay Structural Inadequacy							
Better maintenance	3	4	1	3	1	4	2
Rehabilitation	1	1	1	1	2	2	3
Changing regulations (loads/clearance)	2	3	3	2	4	1	1
Accepting higher risks	4	2	5	5	5	3	-
Reasons for Accepting Functional Inadequacies							
Avoiding traffic disruptions	3	4	5	1	1	4	-
Conservation as a monument	5	2	2	4	5	3	1
Budget constraints	1	3	5	3	3	1	-
Possibility of doubling/extending the bridge	2	1	1	2	2	2	2

[1]code 1 = most used, code 5 = least used

Table 2.8
A Posteriori Estimation of Mean Service Life of Steel and Concrete Bridges According to Belgian Method [45].

Country	Ref. Years	Average Life (and Square Error) in Years		
		Steel	Concrete	
			Reinforced	Prestressed
Great Britain	1977-85	68(18)	72(17)	42(11)
Denmark	1976-88		76(26)	
CH	1980-88		95(31)	
Japan	1967-76 1977-86	47(17) 50(17)	47(15) 57(19)	21(4) 35(9)
Sweden	1984-88	76(21)	73(17)	
SF	1988	75(26)	86(21)	

2.9 Code and Specification Requirements on Durability

Whereas many code and specification requirements may indirectly pertain to durability, there are not many direct requirements to be met in this area. (An exception to this are the fatigue requirements in the design of steel members and connections.) This is somewhat understandable since the primary purpose of codes and specifications is to protect the safety and welfare of the public. And, things pertaining to a structure's durability/ longevity and life cycle cost are not strongly compatible with these objectives. Additionally, codes and specifications typically specify minimum requirements to protect the public and this is not real compatible with achieving good longevity. Unfortunately, companies, municipalities, state agencies, etc. tend to design/construct facilities to meet these minimum code requirements without exceedance. This is probably a valid logic if initial safety and initial cost are the only things of concern. However, if durability/longevity and life-cycle-cost are a high priority, then it is probably not a valid logic to design for minimum requirements to achieve initial satisfactory performance.

In reinforced and prestressed concrete design, durability is directly addressed in codes and specifications via:

1. Concrete ingredience/mix design requirements and limitations
2. Concrete cover requirements

Of course, requirements pertaining to concrete handling, placement, consolidating, finishing, curing, etc. all have impacts on the durability/longevity of the finished product. However, achieving durability/longevity is not the primary intent of these requirements.

Requirements of ACI 318-89 [2] directly pertaining to durability are given in Appendix A. As can be seen, they all relate to concrete ingredients for various adverse exposure conditions, and to concrete cover for rebar and/or prestressing tendons.

Requirements of AASHTO Standard Specifications for Highway Bridges - 1992 [1] which directly pertain to durability of concrete bridges are also given in Appendix A. It is interesting to note that little consideration is given to fatigue in concrete bridges (this is not the case with steel bridges). Section 8.16.8.3 of the AASHTO Specifications, which places a limit on the stress range and use of bends in primary reinforcement, appears to be the sole fatigue requirement.

2.10 Summary and Closure

This literature review focused on the primary sources or causes of bridge deteriorations and what can be done to mitigate these deteriorations. A summary listing of the primary causes of deterioration identified in the review is shown below.

- Age
- ADT/ADTT
- Chemical attack from reactive aggregate and soils
- Rebar and prestressing tendon corrosion
- Concrete deck deterioration due to cracking, scaling, wearing and polishing, and spalling
- Drainage directed onto structural elements
- Faulty joints (either by leaking or clogging)
- Horizontal surfaces where water can stand
- High tire pressures
- Use of studded tires/tire chains
- Lack of sufficient funding
- Scouring of bridge foundations
- Streambed instability
- Improper concrete curing
- Reflective cracking above PCC pavement joints
- Collision damage
- Structural steel corrosion
- Brittle fracture
- Extreme cold temperatures that change material toughness
- Residual stresses in steel members
- Stress concentrations due to geometric discontinuities
- Fatigue stress ratio R
- Poor construction quality

- Poor quality materials
- Fatigue loadings
- Poor drainage
- Poor design details
- Freeze-thaw
- Solar thermal loadings
- Concrete shrinkage
- Improper concrete placing, consolidating and finishing
- Truck overloads
- Deicing salts
- Poor QA/QC programs
- Improper construction sequences
- Improper rebar cover
- Poor maintenance practices
- Poor projection of traffic volume and loads
- Poor projection of number of lane and lane width requirements
- Severe marine environment
- Poor weld joint design
- Welding techniques/shop practices.

A summary listing of actions identified during the literature review to mitigate bridge deteriorations, and thus enhance durability/longevity, is presented below.

- Design quality into bridges
- Correct simple problems quickly
- Increase concrete quality i.e., low water cement ratio, low permeability, high strength, air-entrained, and compatible of ingredients

- Increase depth of rebar cover
- Use epoxy coated rebar
- Install joint seals to stop leaking and debris from entering joints
- Employ thicker decks
- Use shorter simple spans with zero skew
- Have good deck drainage
- Minimize number of joints of eliminate joints
- Design with inspection and maintenance in mind
- Focus concrete construction practices on quality of in-place concrete, and cover over the rebar
- Keep joints clean and clear of debris
- Use rip rap, spur dikes, and improvements to channel to stabilize the channel and to stop scouring of the substructure
- Add redundancy
- Use prefabricated sections
- Hire quality conscious construction contractors
- Have higher prequalification requirements for construction contractors and subcontractors
- Use precautions like reaming bolt holes, smoothing welds, and avoid making multiple passes of welds
- Design steel connections for fatigue loadings
- Avoid placing welds in regions where there are large displacements
- Avoid using small welds on thick plates and vice versa
- Avoid restrained internal sections
- Avoid using stiffeners and cross-bracing
- Keep storage time of prestressing tendons to a minimum

- Eliminate diaphragms in steel girder bridges
- Keep steel painted and concrete sealed
- Use weathering steel

3. BRIDGE CATEGORIES/CLASSIFICATIONS AND DEVELOPMENT DECISIONS

3.1 Bridge Development Sequence and Decisions

As evident from Chapter 2, highway bridges require replacement for a multitude of reasons. The FHWA classifies all of the reasons into 2 basic categories, i.e., structurally deficient and functionality obsolete. Each of these categories will be discussed from a FHWA classification perspective later in this chapter. For the present time let us consider the primary decisions and activities that impact durability/longevity as a bridge evolves from its initial planning stage through the design, construction and operation/maintenance stages. These decisions and activities are briefly discussed below under the subheadings of the ALDOT Bureau responsible for the decisions or activities.

3.1.1 Planning/Design Bureau

Many of the primary selections/decisions which impact future bridge durability/longevity are made at the beginning of the bridge design process by the Planning/Design Bureau. These decisions relate to assessing and establishing design criteria/values for the following:

- Vehicle(s) to Design For (Size & Weight)
- Design ADT/ADTT
- Magnitude & Frequency of Truck Overloads
- River Conditions & Stability (Scour & Meandering Prevention Requirements)
- Flood Flow Conditions (Rates, Velocities, High Water Elevation, etc.)

- Width Requirements (No. of lanes, lane width, shoulder requirements)
- Height & Clearance Requirements
- Safety and Guardrail Requirements
- Site Selection

3.1.2 Bridge Design Bureau

Obviously, many of the decisions which will ultimately impact the bridge's durability/longevity performance will be made by the Bridge Design Bureau. These decisions relate to taking the partial set of design criteria established by the Planning/Design Bureau, supplementing it with other criteria within the Bridge Bureau, and then designing or overseeing the detailed design of the as-to-be-built bridge. The primary decisions made relate to:

- Structural Design Loads (Including Estimations of Foundation Settlements and Thermal Loadings)
- Material and Strength Level Selection
- Selection of Structural Form (Including SS or Continuous)
- Bridge Spans, Expansion/Contraction Joints and Geometry
- Detailed Bridge Member & Joint Design
- Selection of Bridge Joint and Bearing Hardware
- Structural Detailing Requirements

3.1.3 Construction Bureau

Quality and hence durability of a finished product begins with good planning and design. However, this good work may go for naught if it is not implemented/constructed in a quality manner. Construction activities and decisions which have a significant impact on bridge durability are as follows:

- Decision on Construction Company That is Awarded the Contract
- Activities Related to Quality of Materials In-Place
- Construction Sequences
- Construction Methods
- Construction Workmanship
- Time of Year of Construction and Site & Weather Conditions

3.1.4 Maintenance Bureau

Bridges operate in a quite adverse environment, both natural and man-made, and hence, it is essential that they be properly maintained if they are to achieve the desired longevity. This is the responsibility of the Maintenance Bureau. Specific activities required to achieve good bridge durability and longevity are:

- Performance of Quality Periodic Preventative Maintenance
- Performance of Timely and Quality As-Needed Minor Maintenance
- Performance of Timely and Quality As-Needed Bridge Repairs and Rehabilitations
- Conduct Quality and Comprehensive Periodic Bridge Inspections

The following two items may not be under the jurisdiction of the Maintenance Bureau; however, they are activities that should be taken in the operations stage to help maintain our highway and bridge systems.

- Comprehensive use of fixed-site weighing scales and tire pressure monitoring to enforce adherence to bridge load and pavement surface pressure limits.
- Judicious use of portable weigh-in-motion equipment on posted bridges to monitor and enforce compliance with posted loads.

3.2 Bridge Structure Types and Materials

Highway bridges are categorized nationally based on the kind of material of which they are made and/or whether they are simple span (SS) or continuous span (CS). The different material/type of span categories are as follows:

1 Concrete (SS)

2 Concrete continuous

3 Steel (SS)

4 Steel continuous

5 Prestressed concrete (SS)

6 Prestressed concrete continuous

7 Timber

8 Masonry

9 Aluminum, Wrought Iron, or Cast Iron

0 Other

Additionally, the following 23 items categorize bridges based on the predominant type of design and/or type of construction.

01 Slab
02 Stringer/Multibeam or Girder
03 Girder and Floorbeam System
04 Tee Beam
05 Box Beam or Girders - Multiple
06 Box Beam or Girders - Single or Spread
07 Frame
08 Orthotopic
09 Truss - Deck
10 Truss - Thru
11 Arch - Deck
12 Arch - Thru
13 Suspension
14 Staved Girder

15 Movable - Lift
16 Movable - Bascule
17 Movable - Swing
18 Tunnel
19 Culvert
20 Mixed types
21 Segmental Box Girder
22 Channel Beam
00 Other

Both of the above categorizations are applied to the bridge main span(s) and approach span(s). In Alabama, the main spans are normally considered to be the portion of the structure which actually spans a body of water, road, or railroad. The approach spans are the spans that lead up to the main spans.

3.3 Breakdown of Bridge Types and Materials in Alabama

Currently, the dominant highway bridge materials for Alabama are concrete, steel, and prestressed concrete, i.e., the first six categories shown in the first category listing above. And, the dominant bridge structural forms are the first six categories shown in the second listing.

Matrices of the number of the six dominant bridge materials and the six dominant structural forms were generated for current ALDOT bridges. These are shown in Tables 3.1 and 3.2 which give the number of ALDOT bridges that have that type of main span(s) or approach span(s), respectively. The numbers shown in these tables account for over 95 percent of the ALDOT bridges (excluding the approximate 1700 culverts in the state). Table 3.3 shows the percentages of each type of span compared to the total number of bridge spans. For example, concrete slabs represent 2.5 percent of the total state-owned bridges in Alabama, and concrete tee beams make up 27.5 percent of the population. Table 3.3 also gives the total percentage of each type of material for ALDOT bridges. As can be seen in that table, concrete (reinforced concrete) is the most widely used material, and stringer/multi-beam or girder is the most popular design/construction type.

3.4 Bridge Components

Bridges are considered to be composed of 3 major components, i.e.,

- Deck
- Superstructure
- Substructure

Condition rating curves for each of the major components for the entire inventory of ALDOT bridges are given in Fig. 3.1. Major components in turn are composed of subcomponents or areas of condition accessment for purposes of tracking bridge adequacy and performance. Subcomponents under each of the 3 major components are shown in Table 3.4. As inferred in this table, each subcomponent is inspected and given a condition rating. Plots of these ratings versus age for a subset of the ALDOT bridge inventory are presented and discussed in Chapter 5.

Table 3.1
Number of ALDOT Concrete, Steel and Prestressed Concrete Main Span Bridges and No. Main Spans

Material/ Design Category	No. of Main Span Bridges and No. Main Spans[1]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	49 (213)	87 (389)	5 (11)	868 (5473)	4 (17)	0 (0)	1013 (6103)
Concrete Continuous	17 (83)	73 (384)	4 (17)	433 (1751)	10 (34)	2 (10)	539 (2279)
Steel	4 (14)	605 (2957)	14 (82)	0 (0)	1 (3)	0 (0)	624 (3056)
Steel Continuous	0 (0)	641 (3205)	48 (241)	0 (0)	0 (0)	0 (0)	689 (3446)
Prestressed Concrete	9 (71)	212 (1237)	21 (162)	15 (72)	4 (24)	0 (0)	261 (1566)
Prestressed Concrete Continuous	2 (10)	210 (1434)	10 (54)	9 (51)	3 (9)	1 (6)	235 (1564)
Totals	81 (391)	1828 (9606)	102 (567)	1325 (7347)	22 (87)	3 (16)	3361 (18,014)

[1] No. of main span bridges is top entry and number of main spans the bottom.

Table 3.2
Number of ALDOT Concrete, Steel and Prestressed Concrete Bridges with Approach Spans and No. Approach Spans

Material/ Design Category	No. of Bridges with Approach Spans and No. Approach Spans[1]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	8 (441)	44 (333)	3 (55)	246 (1655)	0 (0)	0 (0)	301 (2484)
Concrete Continuous	4 (36)	9 (70)	2 (11)	11 (48)	1 (30)	1 (2)	28 (197)
Steel	0 (0)	153 (1298)	2 (10)	0 (0)	0 (0)	0 (0)	155 (1308)
Steel Continuous	0 (0)	40 (463)	2 (5)	0 (0)	0 (0)	0 (0)	42 (468)
Prestressed Concrete	1 (1)	84 (2275)	3 (9)	6 (26)	1 (10)	0 (0)	95 (2476)
Prestressed Concrete Continuous	0 (0)	37 (978)	3 (41)	2 (45)	0 (0)	0 (0)	42 (1064)
Totals	13 (478)	367 (5417)	15 (131)	265 (1774)	2 (40)	1 (2)	663 (7842)

[1]No. of bridges with approach spans is top entry and number of approach spans the bottom.

Table 3.3
Number of ALDOT Concrete, Steel and Prestressed Concrete Spans (Main Plus Approach)

Material/ Design Category	No. of Spans[1]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	654 (2.5%)	722 (2.8%)	66 (0.3%)	7128 (27.5%)	17 (0.1%)	0 (0%)	8587 (33.2%)
Concrete Continuous	119 (0.5%)	454 (1.7%)	28 (0.1%)	1799 (6.9%)	64 (0.3%)	12 (0.1%)	2476 (9.6%)
Steel	14 (0.1%)	4255 (16.4%)	92 (0.4%)	0 (0%)	3 (0.0%)	0 (0%)	4364 (16.9%)
Steel Continuous	0 (0%)	3668 (14.1%)	251 (1.0%)	0 (0%)	0 (0%)	0 (0%)	3919 (15.1%)
Prestressed Concrete	72 (0.3%)	3512 (13.5%)	171 (0.7%)	98 (0.4%)	34 (0.1%)	0 (0%)	3887 (15.0%)
Prestressed Concrete Continuous	10 (0.1%)	2412 (9.3%)	95 (0.3%)	96 (0.4%)	9 (0.1%)	6 (0.0%)	2628 (10.2%)
Totals	869 (3.5%)	15,023 (57.8%)	703 (2.8%)	9121 (35.2%)	127 (0.6%)	18 (0.1%)	25861 (100%)

[1]No. of spans is top entry and percent of spans is the bottom.

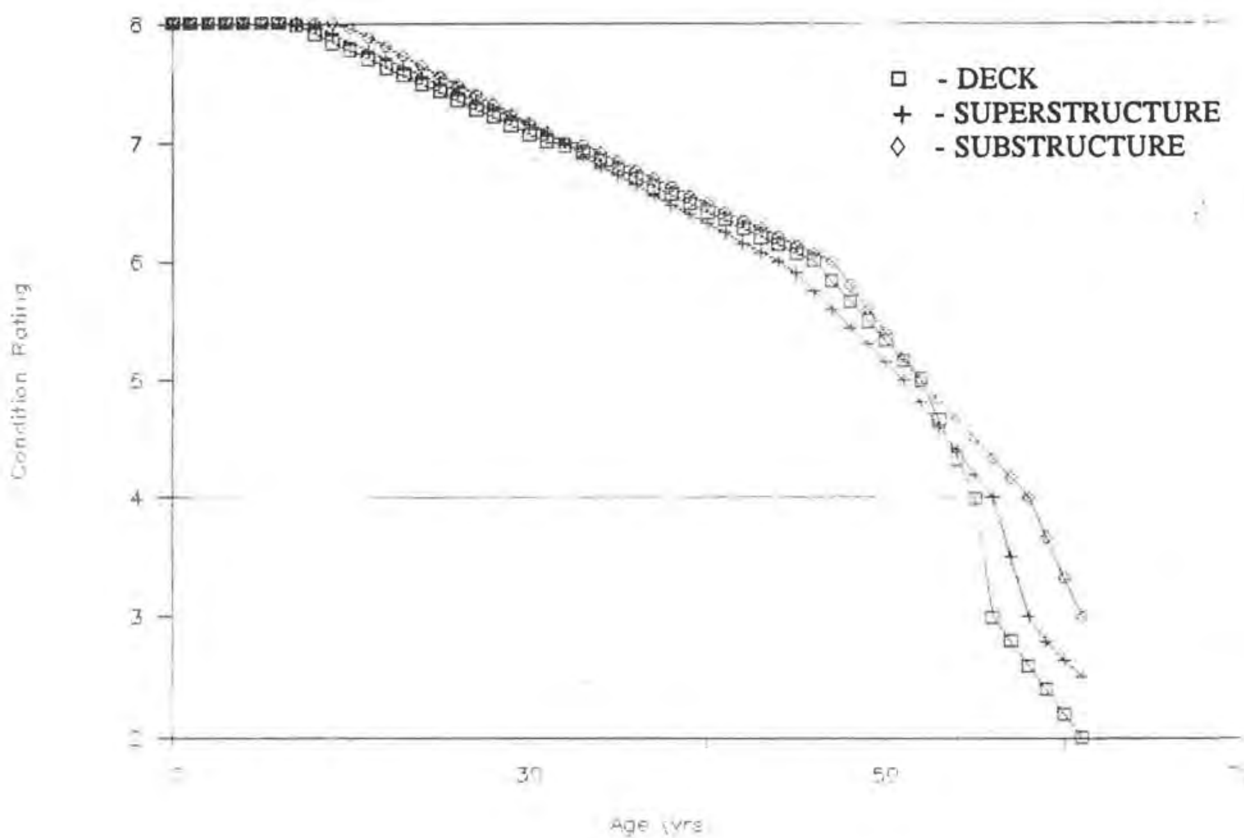


Figure 3.1 Deck, Superstructure and Substructure Condition Rating vs Age Curves for Entire Inventory of ALDOT Bridges (from Ref [60]).

Table 3.4 Bridge Components

Component/Subcomponent		Condition Rating
DECK		
1. Wearing Surface		☐
2. Deck-Structural Condition		☐
3. Curbs		☐
4. Median		☐
5. Sidewalks		☐
6. Railing		☐
7. Paint		☐
8. Drains		☐
9. Lighting Standards		☐
10. Utilities		☐
11. Joint Leakage		☐
12. Expansion Joints/Device		☐
13. Record Elevations		☐
Gutterline @ c Bearings		☐
Gutterline @ c Span		☐
INSPECTOR'S RATING		☐
SUPERSTRUCTURE		
1. Bearing Devices		☐
2. Stringers, Girders and Beams		☐
3. Floor Beams		☐
4. Diaphragms & Cross Frames		☐
5. Trusses-General		☐
-Portals		☐
-Bracing		☐
6. Paint		☐
7. Machinery (Movable Spans)		☐
8. Rivets or Bolts		☐
9. Welds-Cracking		☐
10. Rust		☐
11. Timber Decay, Etc.		☐
12. Concrete Cracking		☐
13. Collision Damage		☐
14. Deflection Under Load		☐
15. Alignment of Members		☐
16. Vibration Under Load		☐
INSPECTOR'S RATING		☐
SUBSTRUCTURE		
1. Abutments	-Caps	☐
	-Wings	☐
	-Backwall	☐
	-Footing	☐
	-Piles	☐
	-Erosion	☐
	-Settlement	☐
2. Piers or Bents	-Caps	☐
	-Columns	☐
	-Footing	☐
	-Piles	☐
	-Scour	☐
	-Settlement	☐
	-Sway Bracing	☐
3. Pile Bents		☐
4. Concrete Cracks of Spalling		☐
5. Steel Corrosion		☐
6. Timber Decay, Etc.		☐
7. Debris on Seats		☐
8. Paint		☐
9. Collision Damage		☐
INSPECTOR'S RATING		☐

3.5 Bridge Condition Inspections and Ratings

Each bridge in the country is inspected and evaluated for adequacy every two years, or more frequently if needed. Over 120 components, subcomponents and other items are inspected and evaluated. The primary items used from the inspection to access the overall adequacy or deficiency of a bridge have been extracted from Ref [16] and are summarized below.

3.5.1 Condition ratings

Condition ratings describe the current condition of the existing, in-place bridge as compared to the as-built condition. Each of the following items (see Ref [16]) are evaluated:

ITEM 58--DECK

ITEM 59--SUPERSTRUCTURE

ITEM 60--SUBSTRUCTURE

ITEM 61--CHANNEL AND CHANNEL PROTECTION

ITEM 62--CULVERTS

The following codes are used as guides in evaluating these items.

Code	Description
N	Not applicable
9	Excellent condition
8	Very good condition - no problems noted
7	Good condition - some minor problems
6	Satisfactory condition - structural elements show some minor deterioration.
5	Fair condition - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.

4 Poor condition - advanced section loss, deterioration, spalling or scour.

3 Serious condition - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.

2 Critical condition - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.

1 Imminent failure condition - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put it back in light service.

0 Failed condition - out of service; beyond corrective action.

3.5.2 Appraisal Ratings

Items 67,68,69,71, and 72 (See Ref [16]) are used to evaluate the bridge in relation to the level of service which it provides to the highway system of which it is a part. The structure is compared to a new one which is built to current design standards for that particular type of road.

ITEM 67 -- STRUCTURAL EVALUATION

ITEM 68 -- DECK GEOMETRY

ITEM 69 -- UNDERCLEARANCES, VERTICAL & HORIZONTAL

ITEM 71 -- WATERWAY ADEQUACY

ITEM 72 -- APPROACH ROADWAY ALIGNMENT

These items are coded with a 1-digit code that indicates the appraisal rating for the item. Codes and their corresponding descriptions are shown below.

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable requiring high priority of corrective action
2	Basically intolerable requiring high priority of replacement
1	This value of code is not used
0	Bridge closed

3.6 Bridge Failure Classifications

Highway bridges eventually "fail" by being classified as either

- Structurally Deficient
- Functionally Obsolete

This requires Condition Ratings (CR) or Appraisal Ratings (AR) as indicated in Fig. 3.2. Values of CR and AR indicated in Fig. 3.2 are as defined in subsections 3.5.1 and 3.5.2 respectively. Only one of the 5 conditions shown under each of the classifications in Fig. 3.2 needs to be violated for the bridge to classified as structurally deficient or functionally obsolete.

It should be noted that any bridge classified as Structurally Deficient is excluded from the Functionally Obsolete category. Also, in order to be considered for either the Structurally

Deficient or Functionally Obsolete classifications, the route must be carried "on" the bridge structure and the bridge be 20 feet in length or longer.

Basically, the Functionally Obsolete mode of bridge failure is as the name implies, i.e., the bridge becomes obsolete relative to the standards of the present time. To the authors, this is closely akin to bridge longevity. The Structurally Deficient mode of failure relates the current condition of the bridge to its as-built condition. To the authors, this speaks more directly to bridge durability performance.

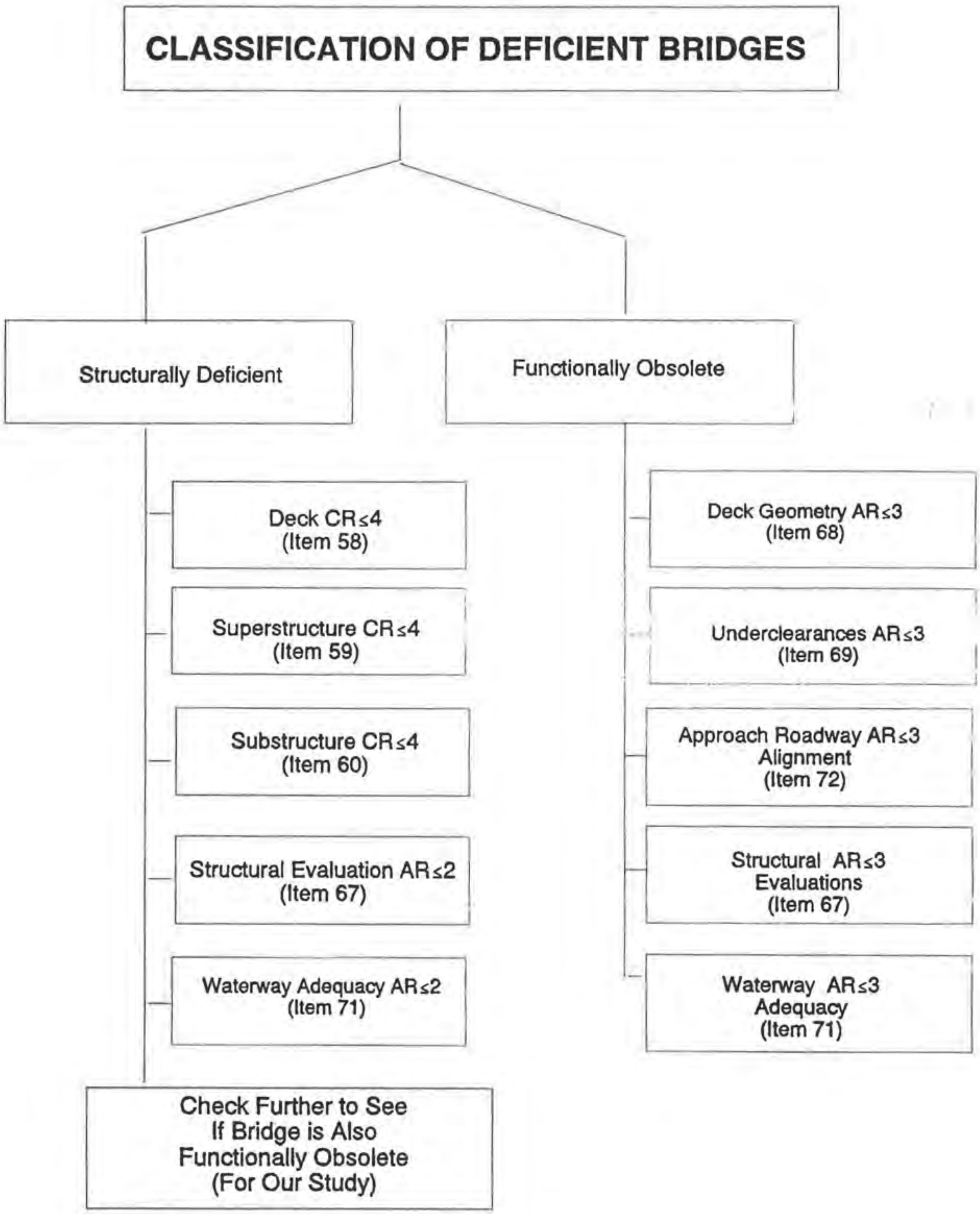


FIGURE 3.2 MODES AND CONDITIONS OF BRIDGE FAILURE

4. RESULTS OF BRIDGE DURABILITY/LONGEVITY SURVEY

4.1 General

In an effort to determine the nature, extent and causes of highway bridge deteriorations in Alabama, and to assess their relative frequency of deterioration, scope and importance, a survey questionnaire on this topic was prepared. State and county engineers, and particularly maintenance engineers in these agencies, throughout the state represent a huge source of knowledge on this topic. Hence, in an effort to tap this source of expertise, the survey questionnaire was sent to this group. It should be noted that this is the only group surveyed, and thus the results are biased toward a "bridge owner's" perspective.

4.2 Survey Questionnaire and Participants

A copy of the survey questionnaire employed along with a sample transmittal letter (a slightly different letter was sent to participants depending on their job position) is shown in Appendix B. The questionnaires were mailed out in December 1992, and were sent to

- All ALDOT Division Engineers (2 questionnaires - one for Division Maintenance Engineer and one for Chief Bridge Inspector)
- ALDOT Bridge, Construction, Maintenance and Materials & Test Bureau Chiefs.
- All County Engineers in Alabama
- Alabama Division of FHWA Bridge Engineer

This translated into 90 questionnaires mailed out, and 46 were completed and returned for a 51 percent response. Results of the survey are presented in the following section.

4.3 Presentation of Survey Results

Results of the survey are summarized on the following pages on a copy of the questionnaire. The number of responses for each question and subquestion are shown beside each response box (□) on the questionnaire, along with the percentage of the responses that the number represents. It should be noted that a few questions were left blank on a few of the questionnaires. In these cases the responses numbered less than 46.

All narrative comments that were offered under <u>Other</u> by the participants are listed under the question where they were offered. And, to enhance readability and analysis of the results, each of the 8 survey questions/results are presented on separate pages.

SURVEY RESPONSE SUMMARY

Survey Questionnaire
to Identify Areas of Focus to Achieve Greater
Bridge Durability and Longevity

Instructions

It is understood that good planning, design, construction and maintenance are all vitally important for a bridge to give excellent service and longevity. What is being asked in the questions below is - "Is additional attention needed (above and beyond current 1992 practices) in various areas to achieve greater bridge durability/longevity?" We ask that you keep this in mind as you answer each question.

Questions

1. I believe that additional attention to bridge longevity in the bridge planning phase (i.e., projection of needed traffic capacity, number of lanes, need for shoulder, flood water levels, etc.) is currently needed to achieve significantly greater bridge durability/longevity.

(82%) 37 ☐ Yes
(18%) 8 ☐ No

Please check the areas below where you think additional attention would pay good dividends in bridge longevity.

(49%) 22 ☐ Planning traffic volume and the number of lanes required
(28%) 12 ☐ Planning lane and shoulder widths
(28%) 12 ☐ Planning horizontal or vertical alignment
(62%) 28 ☐ Planning foundations and clearances for flood frequency and flood water levels
(69%) 31 ☐ Planning for streambed stability (scour)
☐ Other (please specify)

- Anticipate industrial growth; also in rural timber producing counties, anticipate areas of harvest (20 year growth cycle on trees).
- Planning for large volume truck traffic.
- Overhead clearance.
- Type of construction material used and method of construction.

2. I believe that additional attention to bridge durability in the detailed structural design phase (selection of bridge material, design loads, bridge type, spans, continuous vs. simple span, geometry and member sizing, substructure, foundations, etc.) is currently needed to achieve significantly greater bridge durability/longevity.

(90%)	38	☐	Yes
(10%)	4	☐	No

Please check the areas below where you think additional attention would pay good dividends in bridge durability.

(52%)	22	☐	Projection and selection of more appropriate design live loads, i.e., traffic loads (both volume of traffic and loading levels)
(45%)	19	☐	Selection and specification of bridge material type, strength, and other material properties
(33%)	14	☐	Selection of bridge type/structural form
(31%)	13	☐	Selection of continuous vs. simple span
(33%)	14	☐	Selection of straight vs. skewed or horizontally curved girders
(58%)	24	☐	Engineering of bridge, member and connection/joint details to reduce stress concentrations and fatigue damage at fatigue sensitive locations
(43%)	18	☐	Engineering of bridge joints
(31%)	13	☐	Engineering of bridge support bearings
(26%)	11	☐	Engineering of bridge for enhanced constructability
(71%)	30	☐	Engineering of bridge for enhanced future maintainability and inspectability
(29%)	12	☐	Superstructure design for corrosion resistance
(24%)	10	☐	Superstructure design for impact damage from highway traffic
(33%)	14	☐	Bridge deck design for enhanced durability
(31%)	13	☐	Bridge diaphragm to girder connections, transverse X-bracing, etc.
(21%)	9	☐	Substructure structural design
(45%)	19	☐	Substructure design for river meandering
(81%)	34	☐	Substructure design for scour at bridge foundations
(38%)	16	☐	Substructure design for corrosion resistance
(29%)	12	☐	Substructure design for impact damage from river traffic
(69%)	29	☐	Substructure design to resist buildup of debris
		☐	Other (please specify)

- Some structures with high approach fills seem to exhibit a tendency for the abutments to yield to forces created by traffic and soil pressures in the fill and move toward each other. This results in narrowed or closed joints and prevents functioning of expansion devices.

3. I believe that additional attention to bridge durability in the construction phase is currently needed to achieve significantly greater bridge durability/longevity.

(80%) 37 ☐ Yes
(20%) 9 ☐ No

Please check the areas below where you think additional attention would pay good dividends in bridge durability.

(43%) 20 ☐ Achieving the specified bridge material strengths, densities, and other material properties in-place in the constructed bridge.

(65%) 30 ☐ Construction quality to have the "as built" bridge match the "as designed" bridge regarding rebar locations, member sizes, bridge joint/connection details, etc.

(63%) 29 ☐ On site inspection during bridge construction to ensure conformance to design specifications and construction procedures.

☐ Other (please specify)

- Accurate Pile Bearing Calculations.
- I feel we are doing an adequate job of insuring we have a bridge that meets plans & specifications.
- Durability not a problem w/concrete and steel. Functional obsolescence is a problem.
- Training of construction inspectors.
- Training of construction personnel in the site inspection during bridge construction.
- More open communication between maintenance inspection personnel and design personnel in regard to problems with existing bridges.
- Control cracking in concrete structures.
- Continuous on site inspection by a qualified person.
- Performance pay for construction process.

4. I believe that additional attention to bridge durability and longevity in the maintenance phase is currently needed to achieve significantly greater bridge durability/longevity.

(100%)	44	☐	Yes
(0%)	0	☐	No

Please check the areas below where you think additional attention would pay good dividends in bridge longevity.

(16%)	7	☐	More comprehensive and/or thorough biennial inspections
(77%)	34	☐	Acting more quickly on problems identified during inspections
(77%)	34	☐	Greater attention to periodical routine preventive maintenance activities
(75%)	33	☐	Keeping all bridge joints clean and open
(30%)	13	☐	More permanent and effective repair procedures in cases of unplanned bridge damages
(64%)	28	☐	Maintenances of bridge foundation scour control "system"
		☐	Other (please specify)

- Additional funding that is free to be used for extra manpower & equipment to perform needed maintenance work.
- The development of bridge management system to get a better handle on what are problem areas and most cost effective solutions from "do nothing" to replace.
- AHD needs more money to spend on maintenance (personnel too!)
- Keeping bearings & bearing areas clean & free of debris.
- Greater finances for long term maintenance and rehab.
- Sign maintenance and load enforcement.

5. I believe that the type of bridge material is a significant parameter affecting bridge durability performance (for some bridge materials).

(100%)	44	☐	Yes
(0%)	0	☐	No

Please rate the common bridge materials below from 1-10 based on durability/longevity performance (10 = most durable; 1 = least durable). Place a numerical rating in each box.

☐ Reinforced Concrete ($\bar{X}$ = 8.65, S = 1.11, Range = 6-10)

☐ Prestressed Concrete ($\bar{X}$ = 8.63, S = 1.53, Range = 3-10)

☐ Steel ($\bar{X}$ = 7.40, S = 1.37, Range = 5-10)

☐ Timber ($\bar{X}$ = 4.43, S = 1.97, Range = 1-8)

<u>Responder Comments</u>:

10	prestressed concrete	Because of rigid quality control.
10	steel	Because of repairability
8	timber	Life

(Part 1) Yes

(Part 1) Yes, it is of absolute importance.

(Part 2) Reinforced Concrete (I-girders, deck/slabs, box girders)
Prestressed Concrete (I-girders, box beams, channel beams)
Steel (Truss, girder, rolled beam)

Note: Environment plays significant factor. Timber piles in wet/dry zone will not last nearly as long as a timber pile submerged 100% of time.

6. I believe that the type of bridge (combination of structural form and bridge material) is a significant parameter affecting bridge durability performance.

(100%) 42 ☐ Yes
(0%) 0 ☐ No

Please rate the bridge types below from 1-10 regarding durability performance (10 = most durable; 1 = least durable). Place a numerical rating in each box.

☐ Concrete Slab ($\bar{X}$ = 7.93, S = 1.93, Range = 2-10)

☐ Concrete T-Beam ($\bar{X}$ = 8.54, S = 1.60, Range = 3-10)

☐ Steel Rolled Beam/Concrete Deck ($\bar{X}$ = 7.70, S = 1.42, Range = 3-10)

☐ Steel Plate Girder/Concrete Deck ($\bar{X}$ = 7.55, S = 1.30, Range = 5-10)

☐ Prestressed Concrete Girder/Concrete Deck ($\bar{X}$ = 8.92, S = 0.96, Range = 7-10)

☐ Concrete Box Girders ($\bar{X}$ = 7.97, S = 1.77, Range = 2-10)

☐ Prestressed Concrete Box Girders/Concrete Deck ($\bar{X}$ = 8.14, S = 1.42, Range = 5-10)

☐ Steel Box Girders/Concrete Deck ($\bar{X}$ = 7.48, S = 1.57, Range = 4-10)

☐ Steel Truss & Floor System/Concrete Deck ($\bar{X}$ = 6.23, S = 2.04, Range = 1-10)

☐ Timber Truss & Floor System/Timber Deck ($\bar{X}$ = 3.42, S = 1.84, Range = 1-8)

☐ Timber Girders/Timber Deck ($\bar{X}$ = 3.32, S = 2.00, Range = 1-8)

☐ Other (please specify)

- Steel Rolled Beam/Timber Deck
- Steel Girders/Timber Deck
- Reinforced Concrete Deck Girder

7. I believe that certain bridge components (decks, etc.) are quite sensitive to premature deterioration while other components are not.

(100%)	45	☐	Yes
(0%)	0	☐	No

Please rate the bridge components below from 1-10 regarding durability performance (10 = most durable; 1 = least durable). Place a numerical rating in each box.

☐ Deck ($\bar{X}$ = 6.86, S = 2.11, Range = 1-10)

☐ Curb/Sidewalk/Guardrail ($\bar{X}$ = 6.98, S = 2.38, Range = 2-10)

☐ Span Supporting Girders ($\bar{X}$ = 7.78, S = 1.24, Range = 5-10)

☐ Diaphragms and Other Transverse Secondary Members ($\bar{X}$ = 6.95, S = 1.54, Range = 3-10)

☐ Truss Members ($\bar{X}$ = 6.02, S = 1.90, Range = 1-10)

☐ Joint Assemblies ($\bar{X}$ = 5.26, S = 2.08, Range = 1-10)

☐ Bearing Assemblies ($\bar{X}$ = 6.00, S = 1.81, Range = 2-10)

☐ End Abutments ($\bar{X}$ = 6.86, S = 2.18, Range = 1-10)

☐ Intermediate Bent Caps ($\bar{X}$ = 7.88, S = 1.50, Range = 4-10)

☐ Intermediate Bents/Piers/Footings ($\bar{X}$ = 7.02, S = 2.12, Range = 1-10)

☐ Other (please specify)

- Timber piles at water line, steel or steel encased piles at water line in streams with pH problems.
- Deck - Depends on LL deflection, differential deflection, s.s. or continuous, construction quality.
  Bearing Assemblies - Steel (elastomeric=10)
- Piles (Timber, Steel)

8. I believe an important question that should be asked and addressed is the following:

My answer and/or guidance on this question is as follows:

Note: 32 questions and answers were received under this question. To provide some order to these responses they have been grouped into the subareas of planning, design, construction and maintenance below.

Planning:

1. Geometry consistent with traffic volume?

   Single lane dirt roads do not require a mega-bridge.

2. Will all new bridges be designed to take the volume and severity of loads in the year 2042?

   Often we do not take this into account. We have a number of bridges "sound as a dollar" but only 20 ft. wide. They were "state of the art" in the 1950's & 60's but are too narrow in the 1990's. Our philosophy in Dale County 1) Build no wooden structures in this humid climate. 2) Where at all possible, replace secondary bridges with mult. barrel box culverts. They are safer, easier to extend, require less maintenance, and the motorist often is unaware he has crossed a creek. 3) Use precast concrete bridges with steel pilings wherever feasible. They are: 1) much easier and quicker to build 2) much easier to design and 3) usually have better quality control.

3. What will the clearance requirements be for traffic during the life of the structure?

   Liability loss must be added to traffic control structures (rails, signs, end treatments etc.) when comparing additional length (culverts) or width (bridge) costs.

4. Why cannot the State Highway Dept. cooperate with Dept. of Public Safety and furnish more "scale" units? Why not have one for each division (avg. of six counties)?

   No answer given

5. Where do we draw the line between durability enhancement & economy? (Especially for rural areas with lack of construction/maintenance funds)

   A principal research effort should concentrate on the most economic/ durable design development for rural bridge replacement

6. Are we designing our bridges to support future traffic volumes and loads?

   I do not believe enough attention has been given to adequate load capacity and bridge widths in the past. This is one of the main reasons for so many bridge failures today.

Design:

1. I see a problem with the deterioration of concrete. Some cracks show signs of chemical breakdown. Which cracks show a danger, when has concrete deteriorated beyond safe limits, and can mix designs and surface treatments slow deterioration?

   I would like to see non-destructive test developed to determine the soundness of inplace structural concrete.

2. Most of the problems we encounter are timber substructure deterioration & scour and/or debris problems.

   Avoid ground to wood contact; provide adequate waterway opening & eliminate restrictions in the channel.

3. Why do new bridge decks develop transverse cracks during curing? (A common occurrence but no explanation)

   Develop a concrete mix that resists shrinkage cracks or develop and enforce proper curing procedures to prevent shrinkage curing cracks.

4. Which of the different types of bridges discussed in 6 above are better suited to short, long, high bridges, etc.

   I found it difficult to give a concise answer to #6 parts because on site conditions and parameters affect the selection of type, i.e., materials that fit the location.

5. Why not build more bridges with cast in place concrete girders, using continuous spans and open joints at abutments?

   Much fewer maintenance needs.

6. Large traffic volumes appear to cause fatigue in many bridges.

   Additional safety factor in design stage based on traffic volume.

7. Why not design the bridges in the H-25 or higher for additional durability?

   The added cost is more than offset by additional life and stability.

8. Design bridges to be more access to inspectors. Move away from fracture critical designs completely.

   No answer given.

9. What facility would enhance inspection and maintenance of bridges?

   I would like to have additional catwalks and aids to allow better access for inspection and maintenance of bridges.

10. Do we currently consider dynamic loads placed on bridges by approach roadway geometry and also by the percentage of trucks in overall traffic volume when designing bridges?

    Vertical and horizontal curves both in the approach and on the bridge create dynamic forces in the structure that place unplanned forces on various components in the structure. This is enhanced when higher percentages of commercial vehicles use the structures.

11. What type structure along with selection of materials will provide a long life cycle for bridge with minimal maintenance?

    Review existing bridge types along with inspection and maintenance records.

12. Why do we design our bridges for only a 50 year to 75 year lifespan?

    Economics (let the kids pay for replacement). Look at more European designs that have stood the test of time.

13. Why don't we beef up our bridges more and over design them?

    I know the answer to this question is money, but I think in the long run it would pay to spend more money and have less maintenance problems later.

14. I believe that more consideration should be given to 2-span underpasses on major routes.

    This could be accomplished by placing abutments about halfway up the side slopes and would be more economical. This could be accomplished and still meet safety requirements.

15. Can a bridge structure be feasibly designed to withstand today's traffic and loads?

    By increasing the factor of safety in design and using more rigid design this can be achieved.

16. Where do most failures occur regarding bridges built during last 50 years?

    Substructure: Piles, pile bents, caps, stringers. Concrete substructure failures due to rebar oxidation, water leaks, and overload conditions.

<u>Construction</u>: 1. How do we improve quality of inspection of batching concrete, forming and placing embedded items, etc. to ensure quality materials & workmanship?

More quality control by AHD on contractor/supplier/fabricator. More training, more support for inspectors by Central Officer, more/closer supervision of inspectors, more inspectors.

2. Why train inspectors for maintenance inspections and not for construction inspection? Why are bridges accepted with construction errors?

   Train construction inspectors on how & why a bridge should be built as designed. Teach what problem areas to look for, i.e., poor welding, steel placement, sufficient cover on rebar. NBIS inspection before state accepts bridge.

3. Are our construction inspectors provided enough training to adequately perform on complex bridge structures?

   Provide training sessions that address unique construction procedures and maintenance needs.

4. How do you get better quality control on construction projects & workmanship?

   Have better trained inspectors & back them up.

5. What part does quality control (or lack of it) during construction play in maintenance requirements and durability of bridges?

   My experience is that a large part of the deficiencies requiring maintenance could have been avoided if proper quality control and construction practices were followed.

<u>Maintenance</u>:

1. Adequate funding to address deteriorating bridge structures. After 10 years of Federal Bridge Funding, Alabama's counties are status-quo on structurally deficient and functionally obsolete bridge structures.

   Funds must be made available to AL counties to meet minimum standards as per FHWA.

2. Are we paying enough attention to the maintenance & upkeep of our existing bridges?

   Include this as a part of the 2-year bridge inspections: include a form and note if any attention is needed on any bridges that are inspected and explain what is needed such as Ex: Paint, extending pile encasements, correcting abutment scour, etc...

3. Is training of inspection personnel (both construction & maintenance) adequate to achieve greater bridge durability/longevity?

   More "specialized" training is needed and also special emphasis should be placed on selecting personnel capable & <u>willing</u> to access strategic (and sometimes hazardous) areas of the structure.

4. In county government we need adequate funding to meet critical structural problems on some of our county bridges.

   Too much burden and liability on the county engineer when district county comm. controls county finances.

5. Can current staff levels of engineering and road departments (both state & local) keep up with the demands to improve the overall longevity & safety of our bridges?

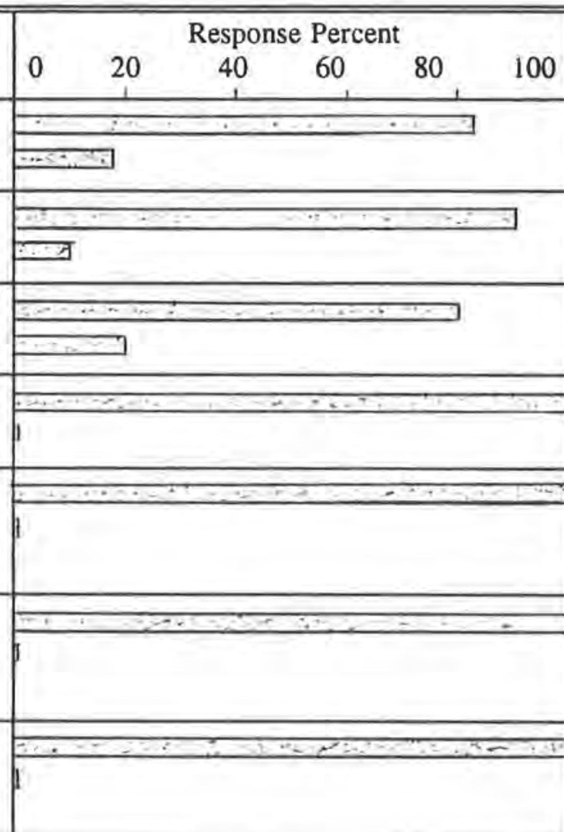
   Funding must be made available which specifically allows the hiring of workers & purchase of equipment. Project funds alone will only further swamp already overloaded staff.

4.4 Analysis of Survey Results

A summary of the responses to the 7 primary questions asked in the survey is provided in graphical form in Table 4.1. The responses reflect that additional attention is needed to enhance durability in all phases of bridge engineering, i.e., planning, structural design, construction and maintenance, with maintenance and structural design getting the strongest endorsement for additional attention. All responders indicated that bridge material and type were very significant to durability performance and that some bridge components were more sensitivity than others.

Table 4.1
Summary of Responses to
Primary Survey Questions.

Question	Response
1. Additional Attention to Durability is Needed in <u>Planning Phase</u>?	Yes No
2. Additional Attention to Durability is Needed in <u>Structural Design Phase</u>?	Yes No
3. Additional Attention to Durability is Needed in <u>Construction Phase</u>?	Yes No
4. Additional Attention to Durability is Needed in <u>Maintenance Phase</u>?	Yes No
5. Bridge Material is a Significant Factor in Bridge Durability Performance?	Yes No
6. Type of Bridge (Structural Form and Material) is a Significant Factor in Bridge Durability Performance?	Yes No
7. Certain Bridge Components are Quite Sensitive to Premature Deterioration While Others Are Not?	Yes No



The subareas under bridge planning, structural design, construction and maintenance where 50 percent or more of the responders viewed that additional attention would pay good dividends in bridge durability/longevity are summarized below.

1. <u>In Bridge Planning</u>

   - Planning for streambed stability/scour (69%)
   - Planning foundations and clearances for floods (62%)
   - Planning traffic volume and number of lanes required (49%)

2. <u>In Bridge Structural Design</u>

   - Substructure design for scour at bridge foundations (81%)
   - Design bridge for enhanced future maintainability and inspectability (71%)
   - Substructure design to resist buildup of debris (69%)
   - Design bridge to reduce stress concentrations and fatigue damage at fatigue sensitive locations (58%)
   - Projection and selection of more appropriate traffic loads (52%)

3. <u>In Bridge Construction</u>

   - Construction quality to have the "as built" bridge match the "as designed" bridge (65%)
   - On site inspection to ensure conformance to design specifications and quality construction procedures (63%)

4. <u>In Bridge Maintenance</u>

   - Greater attention to periodical routine preventive maintenance activities (77%)
   - Acting more quickly on problems identified during inspection (77%)
   - Keeping all bridge joints clean and open (75%)
   - Maintenances of bridge foundation scour control system (64%)

One-hundred (100) percent of the responders indicated that bridge material is a significant parameter in bridge durability performance. They rated the common bridge materials based on durability performance on a scale of 1-10 (10 = most durable and 1 = least durable) as indicated in Figure 4.1. As one would expect, timber is rated relatively low and should probably be viewed as a candidate material for temporary bridges. Reinforced and prestressed concrete were rated about equal and significantly higher than steel as can be seen in Figure 4.1.

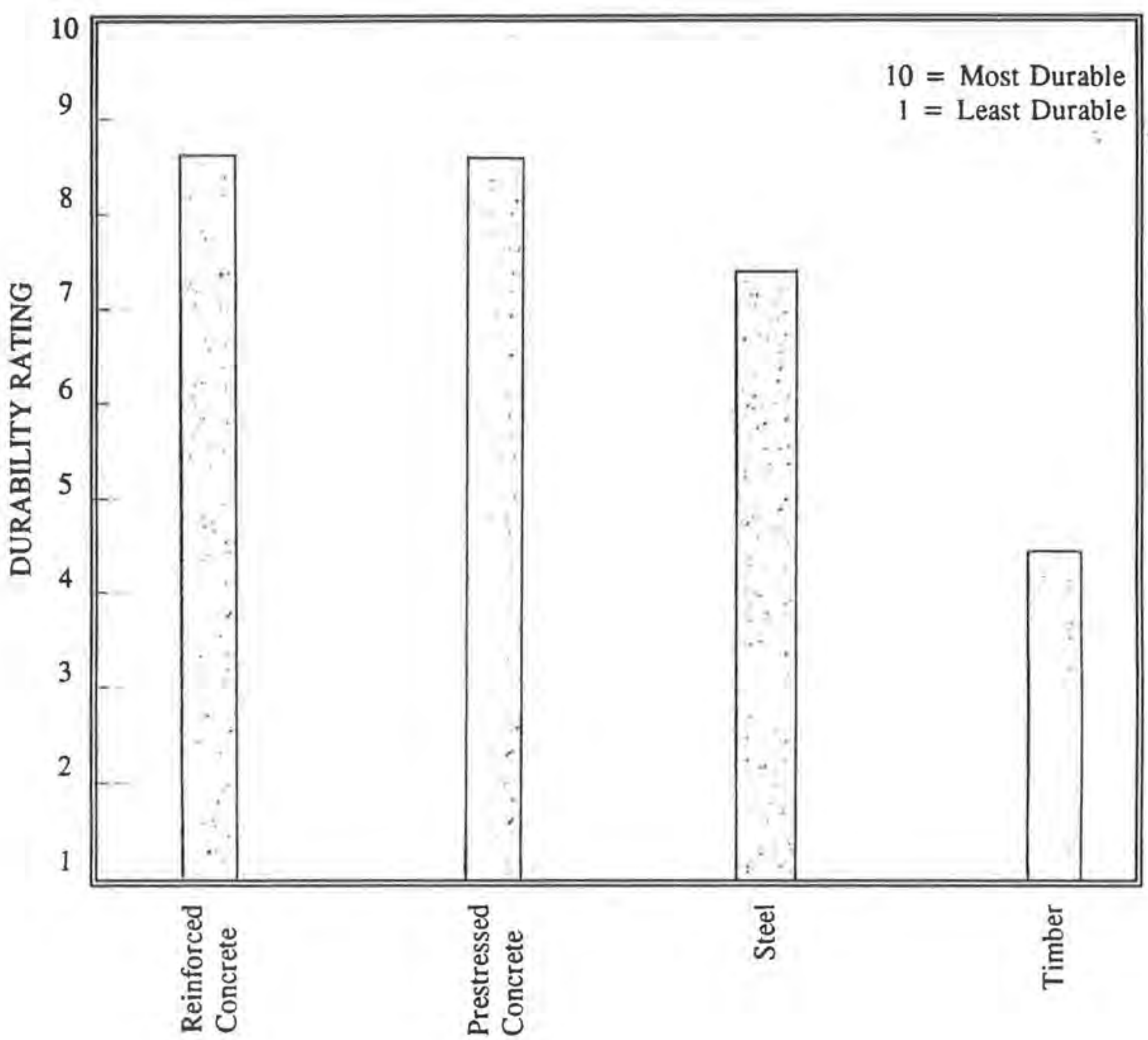


Figure 4.1 Survey Responders Durability Rating of Common Bridge Materials.

No particular structural form stood out as superior from reviewing the responses to survey Question 6 on this topic. However, steel truss bridges were rated low relative to the other structural forms (when temporarily omitting timber bridges) as can be seen in Fig 4.2. Independent of the structural form, prestressed concrete bridges were rated somewhat higher than reinforced concrete in durability performance, and both were rated significantly higher than steel bridges. Again, timber bridges were rated quite low in durability performance.

The survey results on bridge component durability performances are graphically summarized in Fig 4.3. In reviewing these results, two components stick out as giving relative superior durability performances, i.e.,

- Intermediate Bent Caps
- Span Supporting Girders

Three components stick out as giving relative weak performances, i.e.,

- Joint Assemblies
- Bearing Assemblies
- Truss Members

Since trusses and thus truss members are a dying breed of bridge type, the performance of truss members is not of great concern. However, joint and bearing assemblies are, and will continue to be, widely used. The durability of these assemblies are very important in their own right, and additionally, they significantly affect the durability performances of other bridge components.

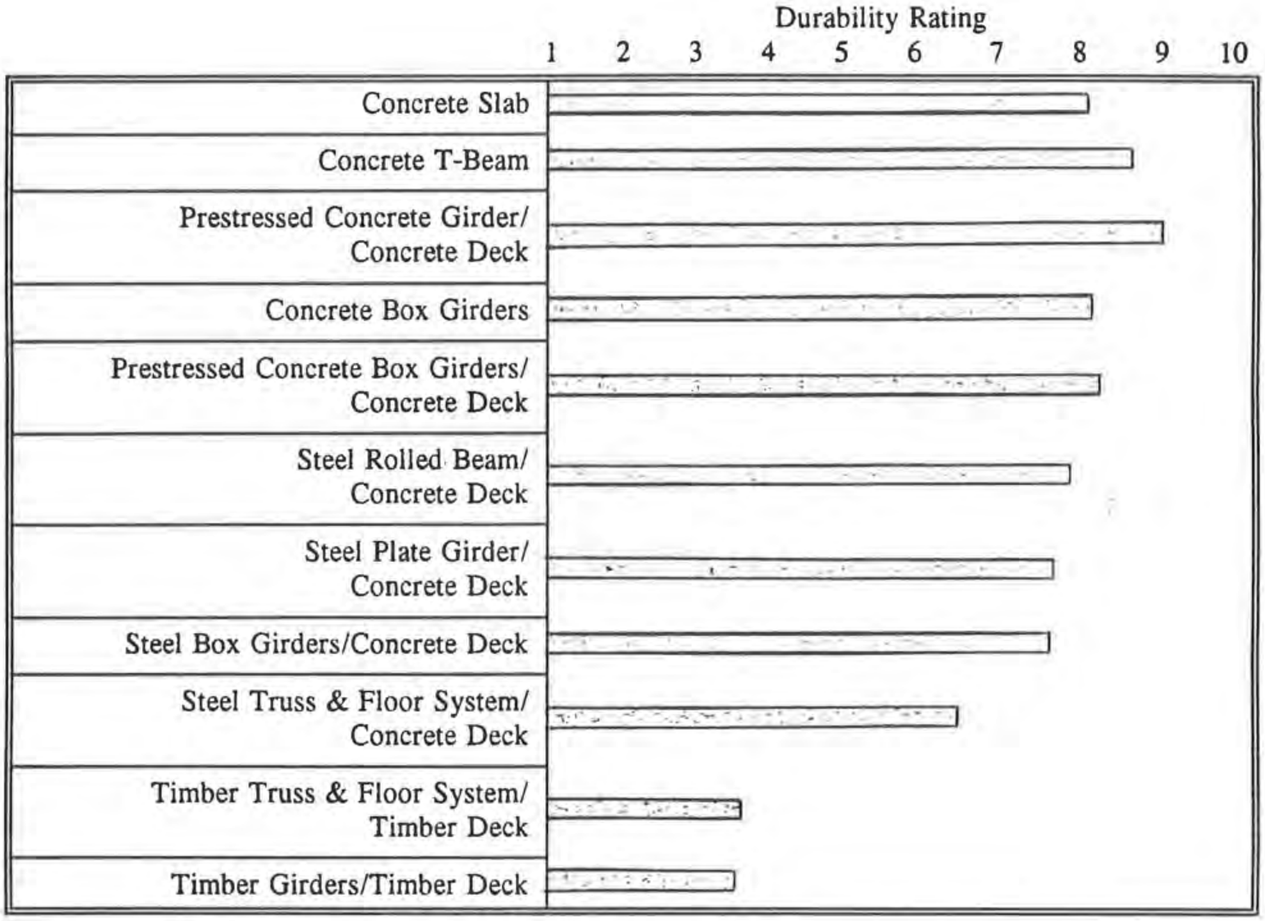


Figure 4.2 Survey Responders Durability Rating of Bridge Structural Form/Material.

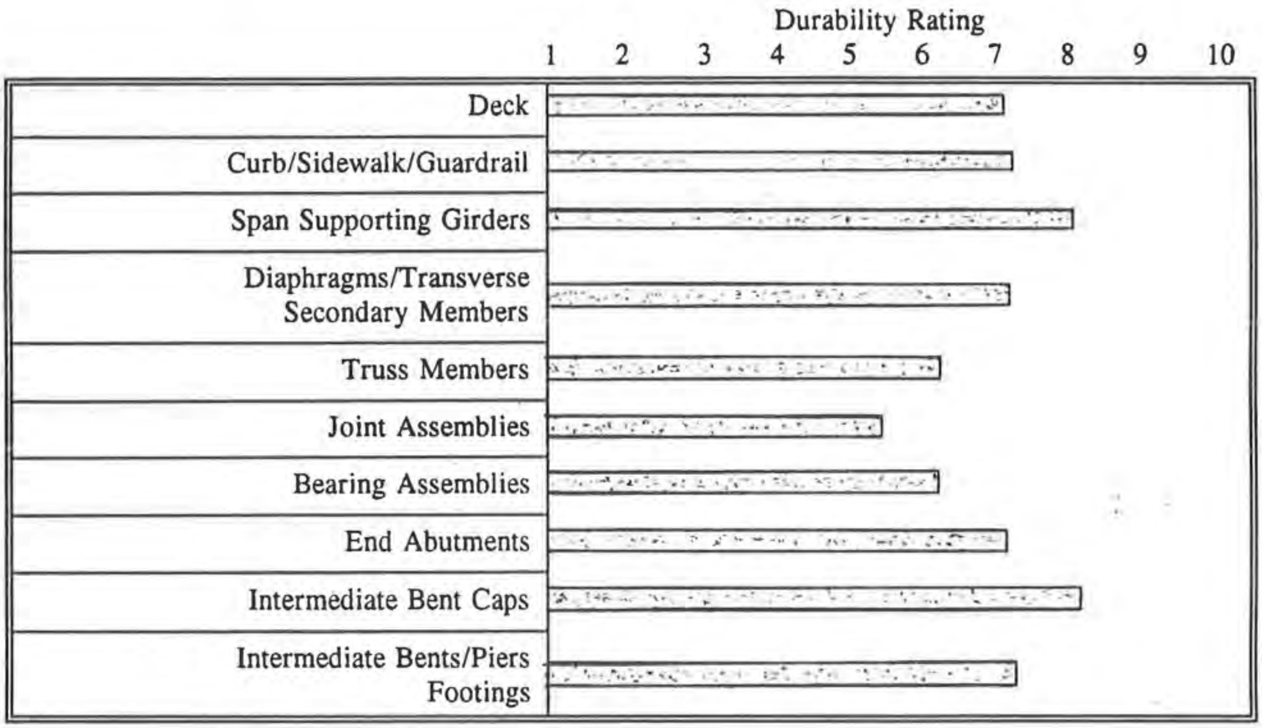


Figure 4.3 Survey Responders Durability Rating of Some Common Bridge Components.

Question 8 of the survey solicited other problems or areas of concern regarding bridge longevity/durability. Thirty-two (32) separate offerings were made and are listed in the previous section. These offerings indicate opportunities for enhancing bridge longevity/durability in the areas listed below. It should be noted that some of the offerings are repeats of those summarized earlier for Questions 1-7. However, a few are new, and it is felt that the repeats express emphasis on those areas repeated.

<u>Planning</u>:

1. Revisit issues related to functional obsolence, i.e., geometry, safety, traffic load, waterway adequacy, requirements, to increase the function life of ALDOT bridges.

2. Revisit decisions on appropriate bridge design life, i.e., 50 years is probably too short.

<u>Design</u>:

1. Emphasize mitigating concrete cracking in bridges.

2. Emphasize mitigating fatigue cracking of steel bridges at diaphragm and transverse bracing locations.

3. Place greater emphasis on improving performance of bridge substructure.

4. Design for larger truck traffic loads, i.e., design for heavier loadings and more loading cycles, and for longer design life.

5. Make designs more "friendly" to inspection and maintenance personnel.

<u>Construction</u>:

1. Develop procedures to achieve more and better inspection by ALDOT personnel during the bridge construction phase.

2. Establish a bridge construction inspection training program for ALDOT personnel.

<u>Maintenance</u>:

1. Provide more funds for bridge maintenance activities.

2. Emphasize preventative maintenance, and act more quickly on problems identified in 2-year bridge inspections.

4.5 Summary and Closure

This survey indicated that the majority of state, county, and maintenance engineers in the state of Alabama feel that additional attention should be given in every phase of a bridge's design/life in order to increase its durability. They feel that the major factors affecting bridge durability are:

- Bridge material
- Structural form
- Appropriate bridge design life (100 years or more)
- Traffic volumes
- Maintenance

Some specific problems and areas that they feel should be given additional attention are:

- Streambed stability
- Flooding
- Difficulty to inspect and maintain due to improper design decisions
- Debris on bridges
- Stress concentrations and fatigue damage
- Construction quality
- Joint assemblies
- Bearing devices

5. RESULTS OF STUDY OF SUBSET OF ALDOT'S HISTORICAL BRIDGE RECORDS

5.1 General

To gain a better understanding of how highway bridges in Alabama are performing in the area of durability/longevity and to access the primary mode and causes of bridge failures, it was logical to review the historical bridge records of the ALDOT. Because of the large number of bridges, it was decided to select a subset of old bridges and to evaluate, to the extent possible, their durability performances. It was important that the subset selected was:

(1) sufficiently old so that they would have records that were appropriate for evaluating durability performances,

(2) of bridge types that were still appropriate for future designs and use by the ALDOT.

Based on the above criteria, a subset of bridges which were recently replaced by the ALDOT was selected for review. In this subset, all timber bridges and culverts were eliminated. A description of the bridge subset used is given in the following section.

5.2 Description of Bridge Data Subset

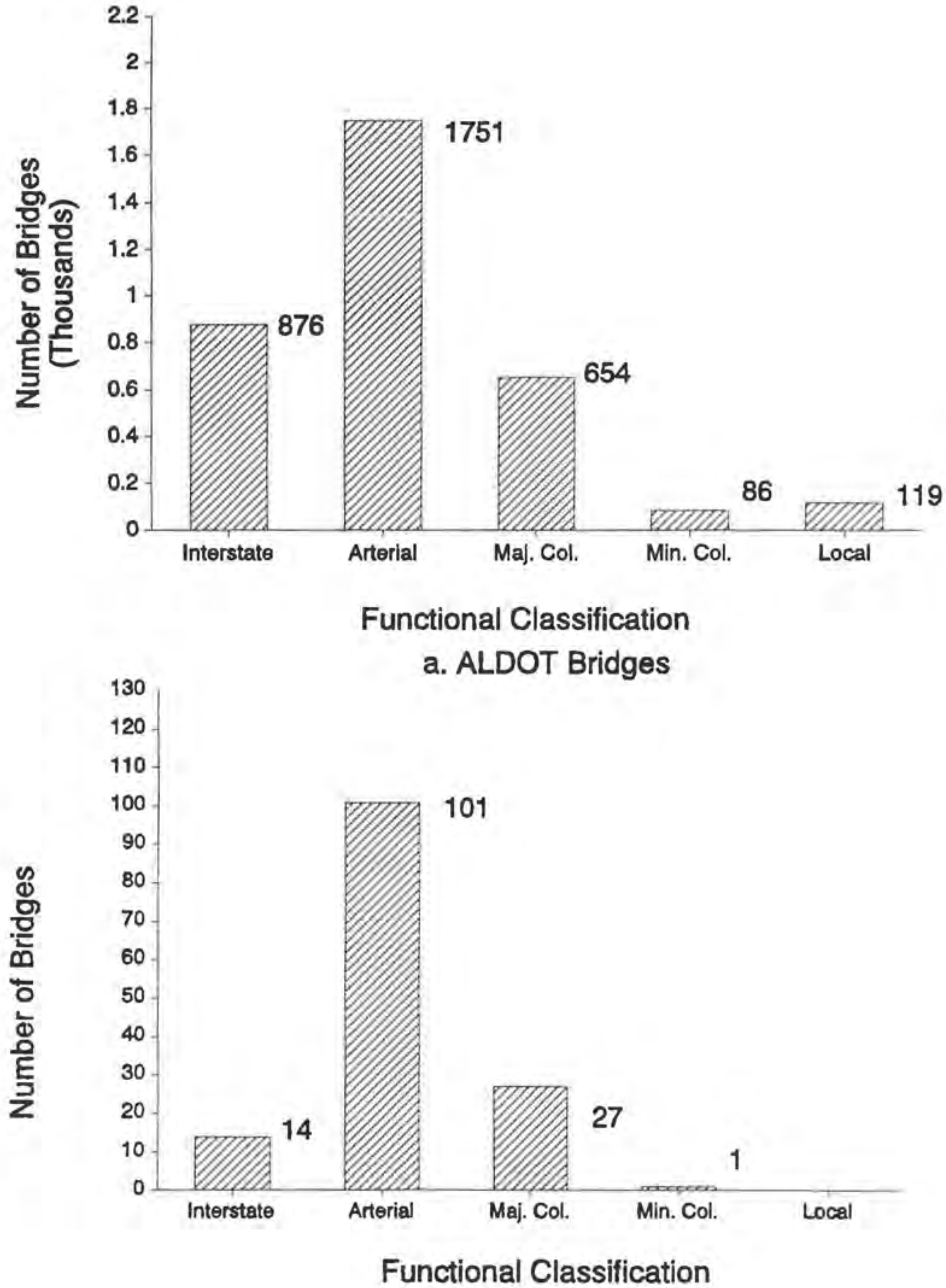
Mr. Fred Conway, ALDOT Bridge Bureau Chief, initially provided a listing of 530 ALDOT bridges which were built/replaced during the 12 years period 1980-1992. Culverts and new bridges (not replacements) were immediately eliminated from the listing as the authors were only interested in how the replaced bridges had performed during their lifespans. Bridge inspection records were then requested for this group of bridges and were furnished by the nine divisions in the state of Alabama.

Timber bridges were also eliminated from the study after it was concluded that ALDOT no longer builds timber bridges. Hence, eliminating these bridges reduced the data set to 166 bridges. Of this group, 23 bridge records were found to be either incomplete or have conflicting information which eliminated those bridges. The final resulting ALDOT bridge sample that was analyzed for durability performance consisted of 143 bridges. Its breakdown by material/structural/construction types is shown in Table 5.1. Hereafter in this report, this dataset will be referred to as AU Subset 143. As seen in Table 5.1, there was only one prestressed concrete stringer bridge in the subset. This is unfortunate since this is currently a popular type of bridge, with a trend pointing to much greater future use. However, data related to the durability performance of prestressed bridges is not readily available as the oldest of these type of bridges are in the 40-45 year range.

For comparative purposes, a breakdown of the AU Subset 143 bridges by functional classification is shown in Figure 5.1, along with a similar breakdown for all ALDOT bridges. This figure indicates that the AU Subset 143 was a fairly representative sample from a functional classification perspective except for interstate bridges. Given the relative young ages of interstate bridges, it is understandable that they would be few in number in the AU Subset 143.

Table 5.1
AU Subset 143 Bridge Types.

Bridge Material & Structure Type	No. Bridges
Concrete Slab	1
Concrete Stringer/Multibeam	2
Concrete Girder or Floor Beam System	1
Concrete Tee Beam	24
Concrete Continuous Slab	1
Concrete Continuous Stringer/Multibeam	1
Concrete Continuous Tee Beam	3
Steel Stringer/Multibeam	80
Steel Continuous Stringer/Multibeam	23
Steel Truss	6
Prestressed Concrete Stringer/Multibeam	1
Total	143



a. ALDOT Bridges

b. AU Subset 143 Bridges

FIGURE 5.1 BREAKDOWN OF ALDOT AND AU SUBSET 143 BRIDGES BY FUNCTIONAL CLASSIFICATION

Information on bridge average daily traffic (ADT) was sought to assess how representative the AU Subset 143 was in this regard and also to assess the importance of this parameter on bridge deterioration. This data was unavailable, and the authors omitted further investigations of this parameter.

Table C-1 in Appendix C provides a listing of the bridges that made up the AU Subset 143. The table lists each bridge by division and gives the bridge's structure number, the year the bridge was built and replaced, and the bridge's age at replacement. The average lifespan of the bridges in the subset was 52 years.

Tables 5.2 - 5.4 provide a breakdown of all ALDOT bridge replacements and new route bridges constructed during the period 1980-1993. In these tables, all bridge replacements were considered, i.e., those replacing timber bridges, culverts, etc. Hence, all of the 530 bridges on the original listing provided by Mr. Conway were included along with those constructed in 1993. This resulted in a total of 639 bridges consisting of 5557 spans. These tables provide good information on the current bridge materials and structural/ construction forms of choice of the ALDOT.

Tables 5.2 - 5.4 are similar to Tables 3.1 - 3.3 which are for almost the entire ALDOT bridge inventory. Comparing Tables 5.4 and 3.3, one can see that the current bridge material trend is away from concrete and steel and toward prestressed concrete (69 percent of the new span construction during 1980-1993 was of prestressed concrete). Also, the trend is away from Tee Beam construction and toward Stringer/Multibeam construction (84 percent of the new span construction during 1980-1993 was of this type).

Table 5.2
Number of ALDOT Concrete, Steel and Prestressed Concrete Main Span Bridges and No. Main Spans in Auburn Subset[1]

Material/ Design Category	No. of Main Span Bridges and No. Main Spans[2]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	2 (7)	25 (98)	2 (6)	0 (0)	1 (4)	0 (0)	30 (115)
Concrete Continuous	1 (1)	23 (148)	4 (17)	29 (108)	0 (0)	1 (2)	58 (276)
Steel	0 (0)	56 (241)	6 (21)	0 (0)	0 (0)	0 (0)	62 (262)
Steel Continuous	0 (0)	136 (531)	10 (32)	0 (0)	0 (0)	0 (0)	146 (563)
Prestressed Concrete	4 (28)	136 (811)	18 (153)	9 (49)	4 (24)	0 (0)	171 (1065)
Prestressed Concrete Continuous	0 (0)	155 (1025)	10 (54)	5 (31)	1 (1)	1 (6)	172 (1117)
Totals	7 (36)	531 (2854)	50 (283)	43 (188)	6 (29)	2 (8)	639 (3398)

[1]All ALDOT bridge replacements and new route bridges constructed during period 1980-1993.
[2]No. of main span bridges is top entry and number of main spans the bottom.

Table 5.3
Number of ALDOT Concrete, Steel and Prestressed Concrete Bridges with Approach Spans and No. Approach Spans in Auburn Subset[1]

Material/ Design Category	No. of Bridges with Approach Spans and No. Approach Spans[2]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	1 (193)	9 (55)	0 (0)	1 (2)	0 (0)	0 (0)	11 (250)
Concrete Continuous	0 (0)	7 (49)	2 (11)	2 (27)	1 (30)	1 (2)	13 (119)
Steel	0 (0)	38 (81)	0 (0)	0 (0)	0 (0)	0 (0)	38 (81)
Steel Continuous	0 (0)	10 (65)	0 (0)	0 (0)	0 (0)	0 (0)	10 (65)
Prestressed Concrete	0 (0)	59 (582)	3 (9)	6 (26)	1 (10)	0 (0)	69 (627)
Prestressed Concrete Continuous	0 (0)	37 (977)	2 (34)	1 (6)	0 (0)	0 (0)	40 (1017)
Totals	1 (193)	160 (1809)	7 (54)	10 (61)	2 (40)	1 (2)	181 (2159)

[1]All ALDOT bridge replacements and new route bridges constructed during period 1980-1993.
[2]No. of bridges with approach spans is top entry and number of approach spans the bottom.

Table 5.4
Number of ALDOT Concrete, Steel and Prestressed Concrete Spans (Main Plus Approach) in Auburn Subset[1]

Material/ Design Category	No. of Spans[2]						
	Slab	Stringer/Multi-Beam or Girder	Girder & Floor-Beam System	Tee Beam	Box Beam/Girder Multiple	Box Beam/Girder Single	Totals
Concrete	200 (3.6%)	153 (2.8%)	6 (0.1%)	2 (0.1%)	4 (0.1%)	0 (0%)	365 (6.7%)
Concrete Continuous	1 (0.0%)	197 (3.6%)	28 (0.5%)	135 (2.4%)	30 (0.1%)	4 (0.1%)	395 (6.7%)
Steel	0 (0%)	322 (5.8%)	21 (0.4%)	0 (0%)	0 (0%)	0 (0%)	300 (6.2%)
Steel Continuous	0 (0%)	596 (10.7%)	32 (0.6%)	0 (0%)	0 (0%)	0 (0%)	628 (11.3%)
Prestressed Concrete	28 (0.5%)	1393 (25.1%)	162 (2.9%)	75 (1.4%)	34 (0.6%)	0 (0%)	1692 (30.5%)
Prestressed Concrete Continuous	0 (0%)	2002 (36.0%)	88 (1.6%)	37 (0.7%)	1 (0.0%)	6 (0.1%)	2034 (38.4%)
Totals	229 (4.1%)	4663 (84.0%)	337 (6.1%)	249 (4.6%)	69 (0.8%)	10 (0.2%)	5557 (100%)

[1]All ALDOT bridge replacements and new route bridges constructed during period 1980-1993.
[2]No. of spans is top entry and percent of total spans is the bottom.

5.3 Modes of Failure in Bridge Subset

As discussed in Section 3.6 of Chapter 3, there are two modes of bridge failure, i.e., structurally deficient and functionally obsolete. Figure 3.1 indicates these modes of failure and the failure categories and values within each mode. These are repeated in Table 5.5 for convenience. For the structurally deficient category, a condition rating of four or lower is considered failed for the deck, superstructure, and substructure, and structural evaluation. The waterway adequacy are considered failed if the bridge appraisal rating drops to two or below. Table 5.5 indicates that the primary structural deficient components for the AU Subset 143 bridges were the deck and substructure, with approximately fifty percent of the bridges failing in these areas.

A bridge would be considered a functionally obsolete failure if its deck geometry, underclearance, approach alignment, structural evaluation, or its waterway adequacy had an appraisal rating of three or lower. Ninety percent of the bridges in the AU Subset 143 data set failed because of their deck geometry, and fifty-two percent failed because of their structural evaluation. It should be noted in Table 5.5 that the number of bridges in each failure category varies because the condition or appraisal rating in each category for each of the 143 bridges was not available to the authors.

Even though Table 5.5 indicates that most of the bridge failures in AU Subset 143 were due to functional obsolence, the table indicates a high percentage of structurally deficient failures as well. This, along with the data shown in Figures 5.7-5.39, indicates that even if the functional obsolence problems were eliminated, the bridges would still have failed at a relatively young age due to structural deficiencies.

Table 5.5
Modes of Failure for
AU Subset 143 Bridges.

Failure Category		No. of Failed Bridges	Total No. of Bridges	Percent Failure
Structurally Deficient	Deck CR $\leq$ 4	72	143	50
	Superstructure CR $\leq$ 4	42	143	29
	Substructure CR $\leq$ 4	70	143	49
	Structural Evaluation AR $\leq$ 2	41	109	38
	Waterway Adequacy AR $\leq$ 2	1	101	1
Functionally Obsolete	Deck Geometry AR $\leq$ 3	103	115	90
	Underclearance AR $\leq$ 3	3	13	23
	Approach Alignment AR $\leq$ 3	19	143	13
	Structural Evaluation AR $\leq$ 3	57	109	52
	Waterway Adequacy AR $\leq$ 3	1	101	1

5.4 Durability Performances of the Major Bridge Components

The major bridge components evaluated for durability performance were the deck, superstructure, and substructure. Condition Rating (CR) versus Age performance plots of these three components are given in Figure 5.2. All three components show mild deterioration at the beginningof their lives, and very significant deterioration later in the life of the bridges. It should be noted that a CR of 9 is given to each bridge/component when it is first built and indicates a new condition or a condition of excellence. Standard practice in the ALDOT is to automatically lower the CR of all components to 8 after the first year in operation. Components continue to be rated an 8 until deteriorations cause the ratings to go down. The solid line, at a CR value of 4, in Figure 5.2 indicates failure of the components.

Separate plots for each major bridge component are shown in Figures 5.3-5.5. Superimposed on these plots are the performance curves for all ALDOT bridges (extracted from Figure 3.1 in Chapter 3). An examination of Figure 5.3 reveals that the AU Subset bridges exhibited early deterioration, beginning around an age of 10 years, but did not show a significant deterioration rate until an age of 39 years. The deterioration rate for the decks of the AU Subset of bridges was 0.21. Figure 5.4 shows the superstructures having significant deterioration at an age of 40 years with a deterioration rate of 0.13. The substructures began showing significant deterioration at an age of 38 years with a deterioration rate of 0.18. All three bridge components for the AU Subset failed at approximately the same age as the complete ALDOT bridge dataset.

Of the three components, the decks performed the worst. This is as expected because of the adverse environment in which the deck exists. It is also good because the deck is probably the easiest and cheapest component to replace. The superstructures performed the best of the three components. It had higher final condition ratings and lower deterioration

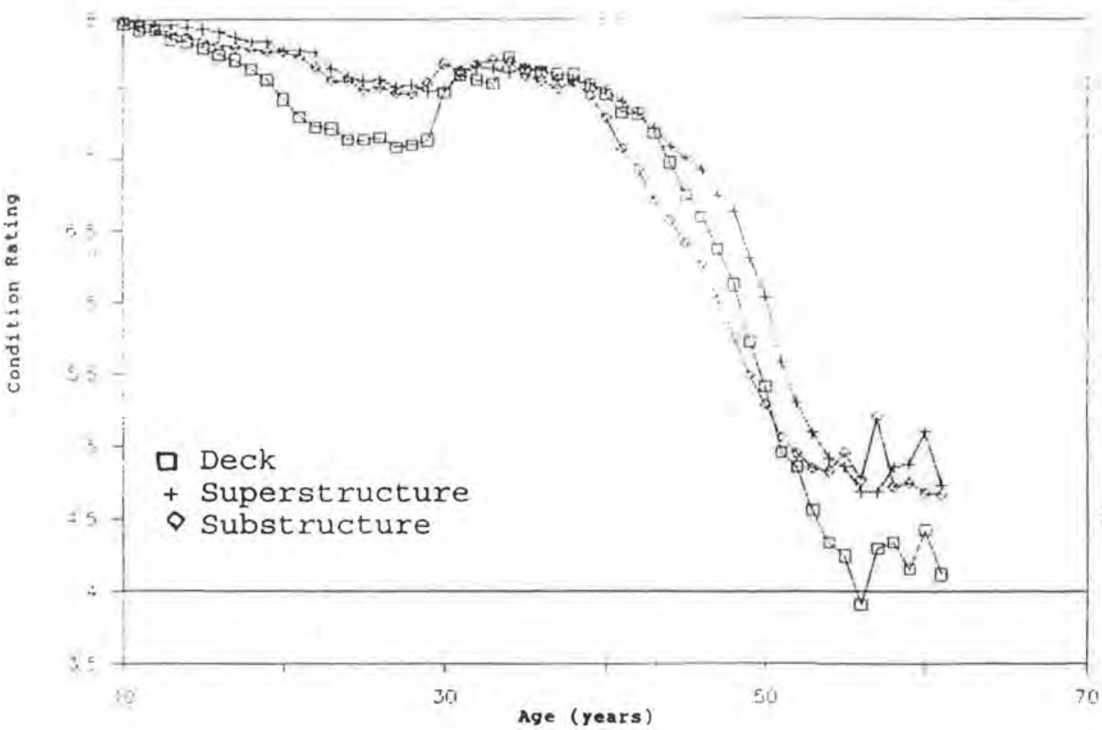


Figure 5.2 Deck, Superstructure, and Substructure CR vs. Age For AU Subset

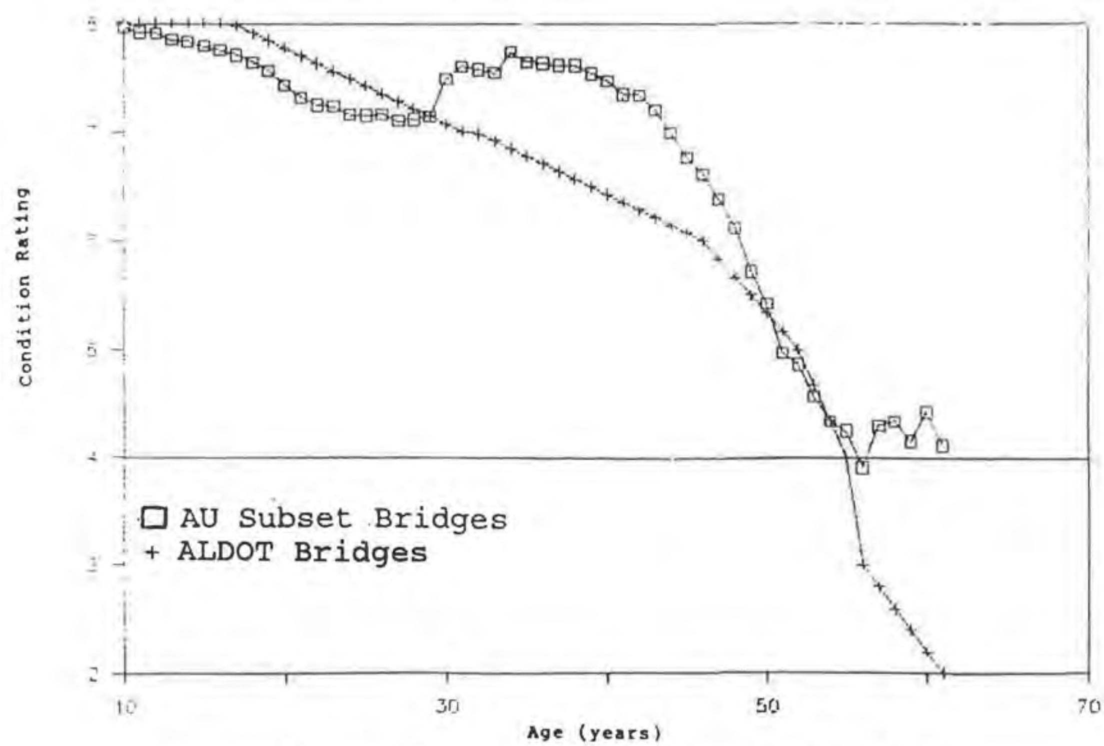


Figure 5.3 Deck CR vs. Age For AU Subset and ALDOT Bridges

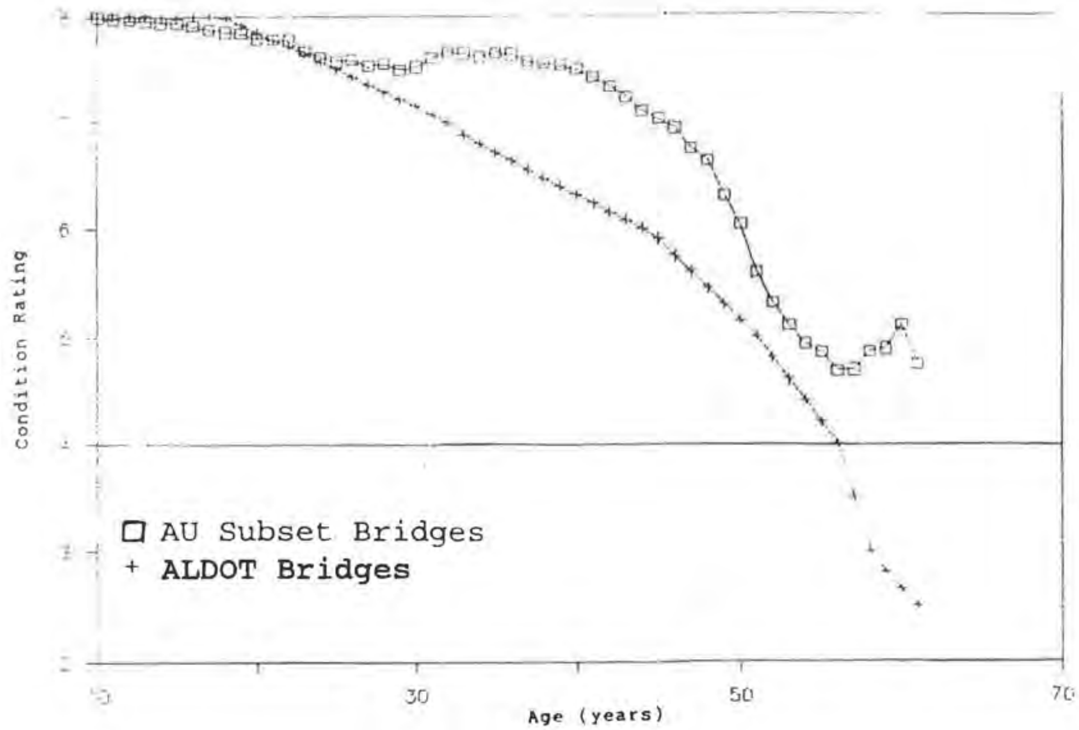


Figure 5.4 Superstructure CR vs. Age For AU Subset and ALDOT Bridges

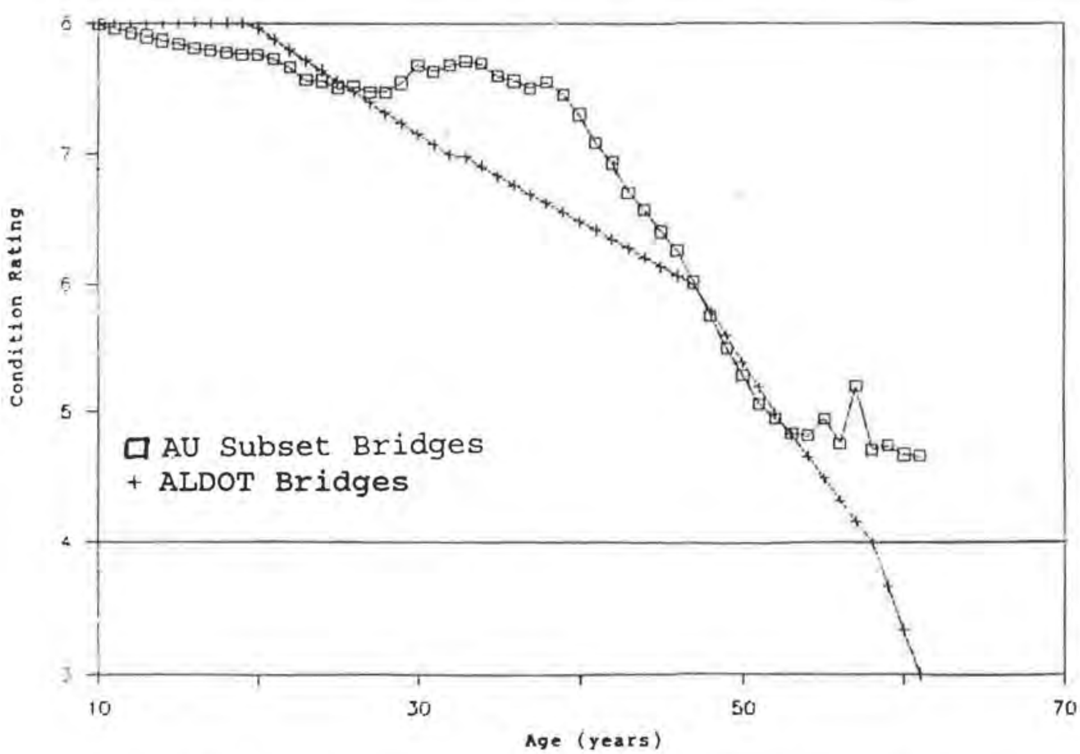


Figure 5.5 Substructure CR vs. Age For AU Subset and ALDOT Bridges

rates. The performance of the substructures fell between the decks and the superstructures. However, overall, all three components exhibited almost identical deterioration curves/ durability performances. This is readily apparent for the AU Subset of bridges in Figure 5.2 and for all of the ALDOT bridges as indicated in Figure 3.1.

Early in this study, before the AU dataset was finalized to that shown in Table 5.1, 153 ALDOT bridges records were examined for major bridge component performances based on functional classification of inventory route. Condition ratings for the deck, superstructure, and substructure were recorded for each of the 153 bridges. These condition ratings represent the final condition the bridges were in when they were replaced. Tables 5.6-5.8 provide the condition ratings for these three major components based on the bridges functional classifications of interstate, arterial, majorcollector, and minor collector. These tables also show the total number of bridges and the percent of bridges that were structurally deficient.

Tables 5.6-5.8 indicate that interstate bridges became structurally deficient because of deterioration of their decks. These data appear reasonable because of the relative newness of interstate bridges. Arterial bridges represent both the majority of bridges in the subset, and the majority of bridges that are structurally deficient. Each component of arterial bridges has the highest percent of deficiency compared to the other functional classes (see Tables 5.6-5.8). Sixty percent of the arterial bridges are structurally deficient because of their deck. Forty-three percent of these bridges are structurally deficient because of their superstructure, and seventy percent of the bridges are structurally deficient based on their substructure condition. Major collector bridges had fifty-five percent that were structurally deficient because of their deck, and thirty-three percent and sixty-three percent were structurally deficient based on their superstructure and substructure, respectfully. Comparing the performances of bridges in arterial vs. major collector highway systems, there is a 5-10 larger structurally deficient percentage for bridges in the arterial system. This higher percentage is probably directly attributable to larger ADTs, and particularly larger ADTTs, for the arterial bridges.

It can be noted in Table 5.6 that structural deficiency ratings based on deck condition ratings for all functional classification of bridges were large. This was even the case for interstate bridges which are relatively new. It is speculated that the high volume of traffic, in general, and high volume of truck traffic, in particular, along with the adverse natural environment exposure of decks, on interstate bridges were sufficient to cause relatively rapid deck deteriorations. Deck deterioration propagates throughout the bridge and leads to superstructure deterioration and, in turn, to substructure deterioration.

For arterial and major collector highway system bridges, the major bridge component showing the greatest deterioration and structural deficiency is the substructure. It is speculated that, for bridges in these systems, what goes under the bridge and the foundation setting of the bridge, may be more important than what goes over the bridge. Of course, substructure deterioration causes an adverse impact on the bridge superstructure and deck with time. The high percentage of substructure deterioration was probably due to river/scour conditions and/or foundation settlements and movements under loads.

Overall, in this bridge subset, sixty-three percent of the bridges were rated structurally deficient because of their substructure condition, fifty-six percent because of their deck condition and thirty-seven percent because of their superstructure condition. Substructure and deck structural deficiency percentages are particularly high and indicate that bridge designers should concentrate on these two major components to enhance the durability/longevity of future bridges. Also, the data appears to indicate that higher volumes of traffic, particularly higher ADTT leads to greater structural deterioration and this should be addressed in future designs.

Table 5.6
Deck Condition Rating by Functional Classification of Inventory Route
AU Subset.

Condition Rating	Function Classification				
	Interstate	Arterial	Major Collector	Minor Collector	Total
0	0	0	0	0	0
2	0	2	0	0	2
3	1	20	2	0	23
4	3	42	16	0	61
5	1	26	10	1	38
6	7	13	4	0	24
7	0	3	1	0	4
8	0	1	0	0	1
9	0	0	0	0	0
Total No.	12	107	33	1	153
Total No. Structurally Deficient	4	64	18	0	86
Percent Structurally Deficient	33.3	59.8	54.5	0	56.2

Table 5.7
Superstructure Condition Rating by Functional Classification of Inventory Route AU Subset.

Condition Rating	Function Classification				
	Interstate	Arterial	Major Collector	Minor Collector	Total
0	0	0	0	0	0
2	0	1	0	0	1
3	0	12	3	0	15
4	0	33	8	0	41
5	2	29	11	1	43
6	7	29	6	0	42
7	3	2	5	0	10
8	0	1	0	0	1
9	0	0	0	0	0
Total No.	12	107	33	1	153
Total No. Structurally Deficient	0	46	11	0	57
Percent Structurally Deficient	0	43.0	33.3	0	37.3

Table 5.8
Substructure Condition Rating by Functional Classification of Inventory Route AU Subset.

Condition Rating	Function Classification				
	Interstate	Arterial	Major Collector	Minor Collector	Total
0	0	0	0	0	0
2	0	2	0	0	2
3	0	26	9	0	35
4	0	47	12	0	59
5	1	22	8	1	32
6	2	8	2	0	12
7	7	1	1	0	9
8	2	1	1	0	4
9	0	0	0	0	0
Total No.	12	107	33	1	153
Total No. Structurally Deficient	0	75	21	0	96
Percent Structurally Deficient	0	70.1	63.6	0	62.7

5.5 Durability Performances of Bridge Subcomponents

Condition Rating (CR) vs Age curves are shown in Figures 5.7-5.39 for a subset of the AU Subset 143 bridges. More specifically, these figures show plots of CR vs Age for various major components and subcomponents for concrete tee beam bridges (Code No. 104), steel stringer/multibeam bridges (Code No. 302), and steel continuous stringer/multibeam bridges (Code No. 402). These particular bridge materials and structural/construction forms were dictated by the AU data set as can be seen in Table 5.1. It is unfortunate that these material types are currently being largely replaced by prestressed concrete bridges (see Section 5.2). However, a comparison of the durability performances of these bridge types still provide very helpful information toward enhancing the durability performances of future bridges. For example, all bridge decks are still concrete, and the deck subcomponents remain the same as do the substructure subcomponents. Even for the bridge superstructures, many of the subcomponents evaluated in Figure 5.7-5.39 remain the same for the new prestressed concrete bridges.

In evaluating durability performances of the various components and subcomponents, the emphasis was placed on the particular component rather than the type of bridge. This was done because it was felt that the key to achieving greater bridge durability is to achieve greater durability of the poorer performing subcomponents of the bridge. That is, the "weak link" will tend to dictate the overall bridge durability performance. Consistent with emphasizing component and subcomponent performances, and for convenience in accessing the performances, the CR vs Age data for the 104, 302, and 402 bridges for each component/subcomponent are shown in the same figure in Figures 5.7-5.39. In a few cases, data was not available for one of the bridge types and thus these CR vs Age plots were left blank. Figures 5.7-5.14 are for the bridge decks and subcomponents, Figure 5.15-5.26 are for the bridge superstructures and subcomponents, and Figures 5.27-5.39 are for the bridge substructures and subcomponents.

Recall that if a bridge component condition rating falls to a level of four or lower, it is considered to have failed. Hence, a solid line is shown on each graph of Figures 5.7-5.39 to indicate where failure occurs.

It should be noted that in plotting the CR vs Age curves for the various bridge components, the authors chose to evaluate the average CR for each year of age by dividing the sum of the CR's for that year by the number of bridges contributing to the sum for that year. That is, as the bridges aged and had to be replaced, they were eliminated from further consideration and only the remaining bridges were utilized in the averaging process. This was felt to be appropriate to avoid confusing and misleading abrupt drops in average CRs resulting from bridges falling out of the data base with age. It was also necessary because of cases of missing data on some bridge CR items, and thus the average values evaluated were for the subset of bridges where that CR item value was given.

Unfortunately, this averaging procedure typically resulted in a leveling-off portion of the CR vs Age curve and then an upswing of the curve at its end, i.e., at the high age values (see Figure 5.6). The cause of this leveling-off and upswing at the end is due to the better performing bridges continuing in service and thus providing biased average CR values. For example, near 60+ years of age, two bridges of an original sample of 22 may remain in service with a CR of 7 each. By the authors' averaging procedure, this would result in a CR of 7 and cause the CR curve to go up at the higher age levels as indicated in Figure 5.6. At best, inclusion of these points confuses the plot, and at worst, it could cause erroneous conclusions. Hence, these points have been eliminated in Figures 5.7 - 5.39. It should be noted, that the high CR of these better performing bridges were included in the downward sloping portion of the CR vs Age curves and are appropriately helping to lower the average deterioration rates for the various curves. Also, in fitting an approximate deterioration curve by "eye" to the data, the leveled-off data points at the high-age levels were also excluded from consideration because of their bias as indicated above.

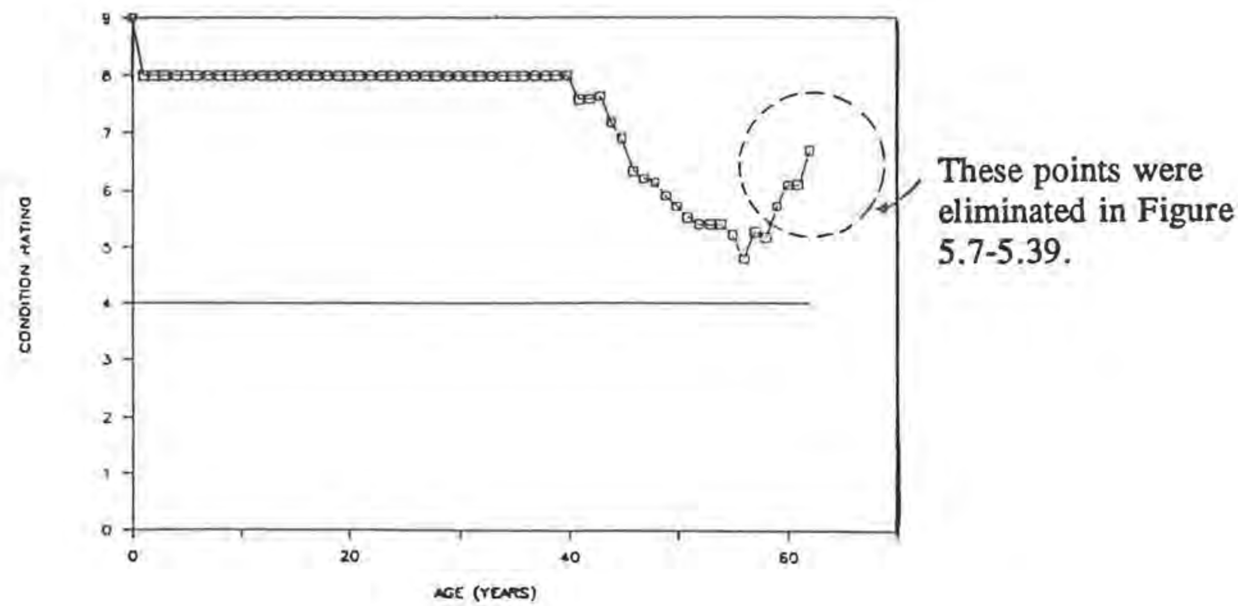


Figure 5.6 Typical Bridge Component Condition Rating (CR) vs Age Curve

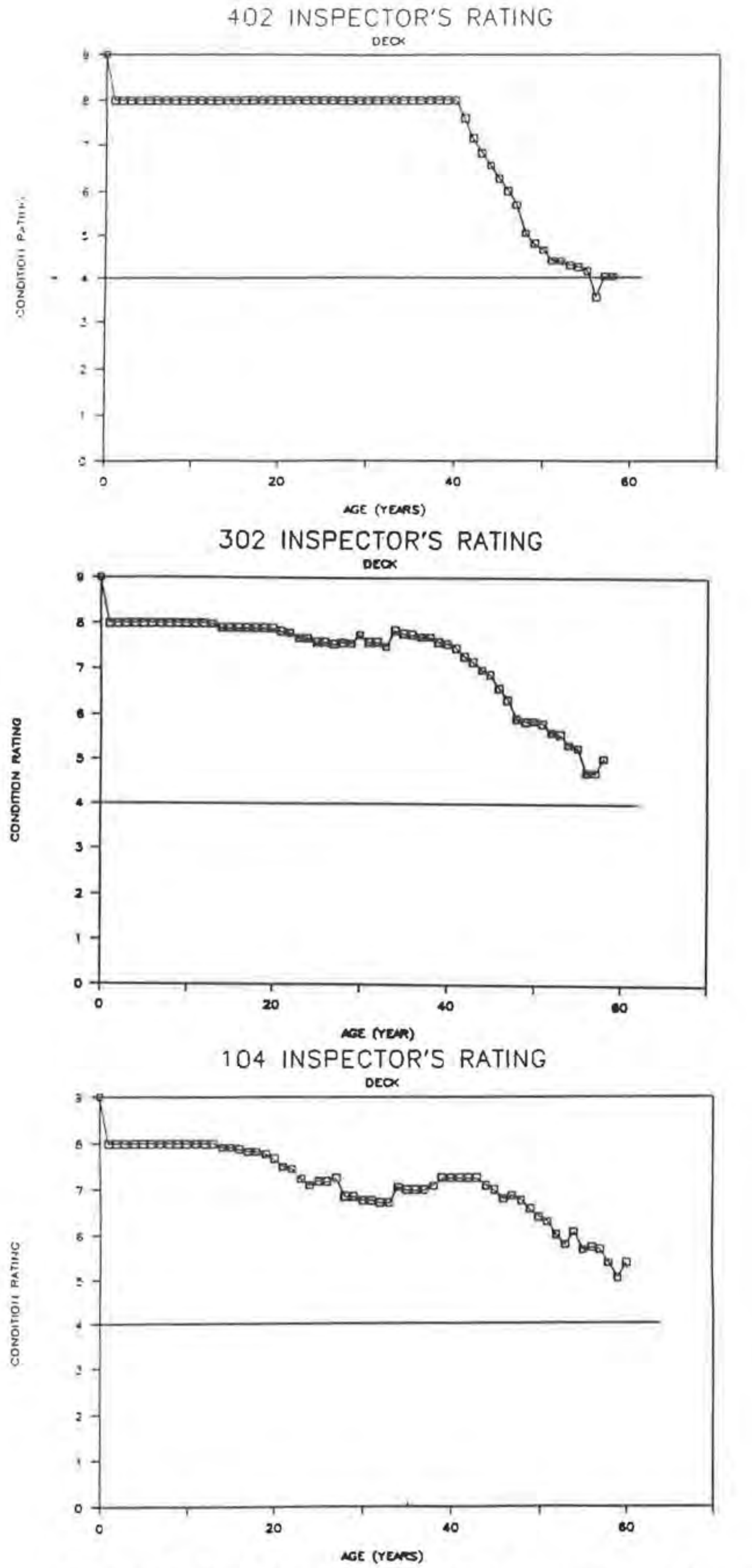


Figure 5.7 Deck Inspector's Condition Rating vs Age Curves for 104, 302, 402 Bridges

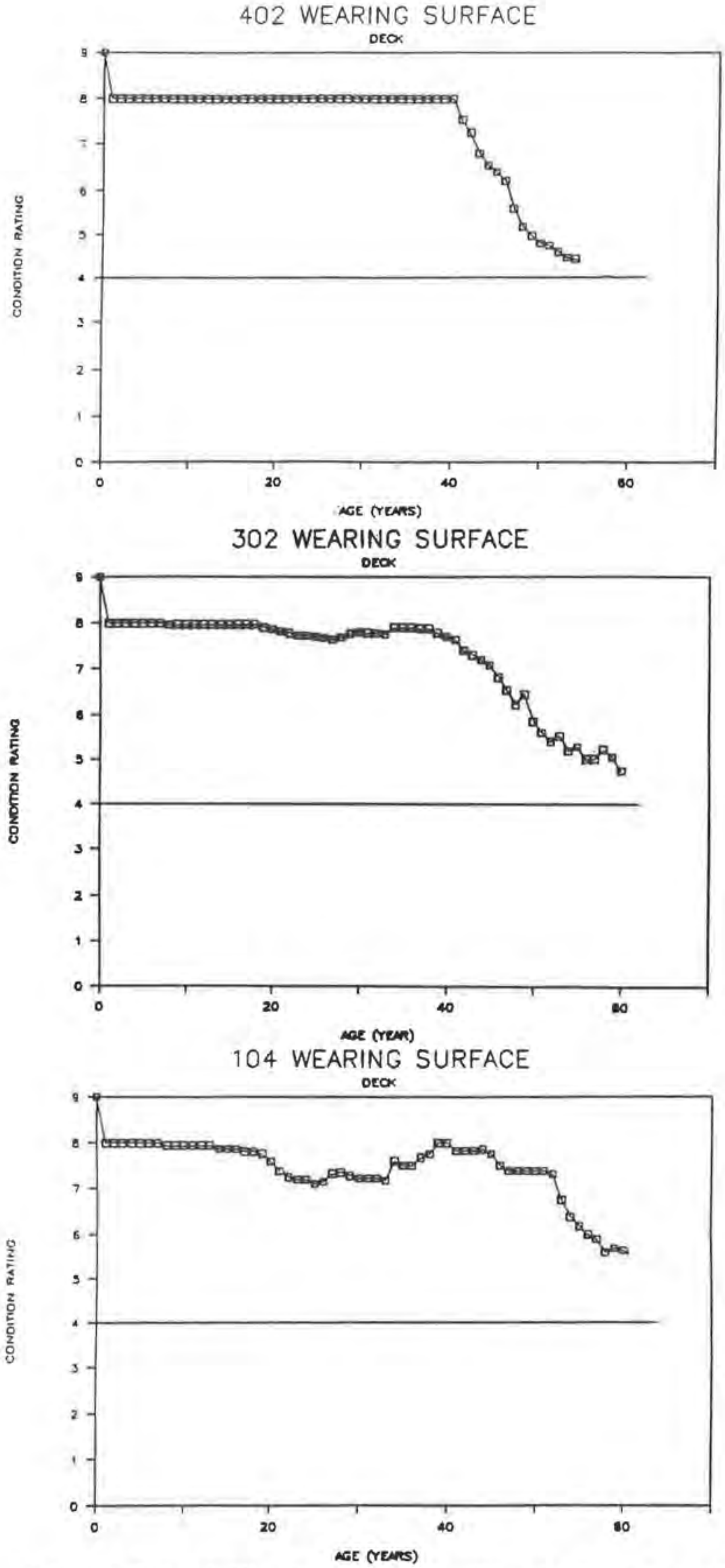


Figure 5.8 Deck Wearing Surface Condition Rating vs Age Curves for 104, 302, 402 Bridges

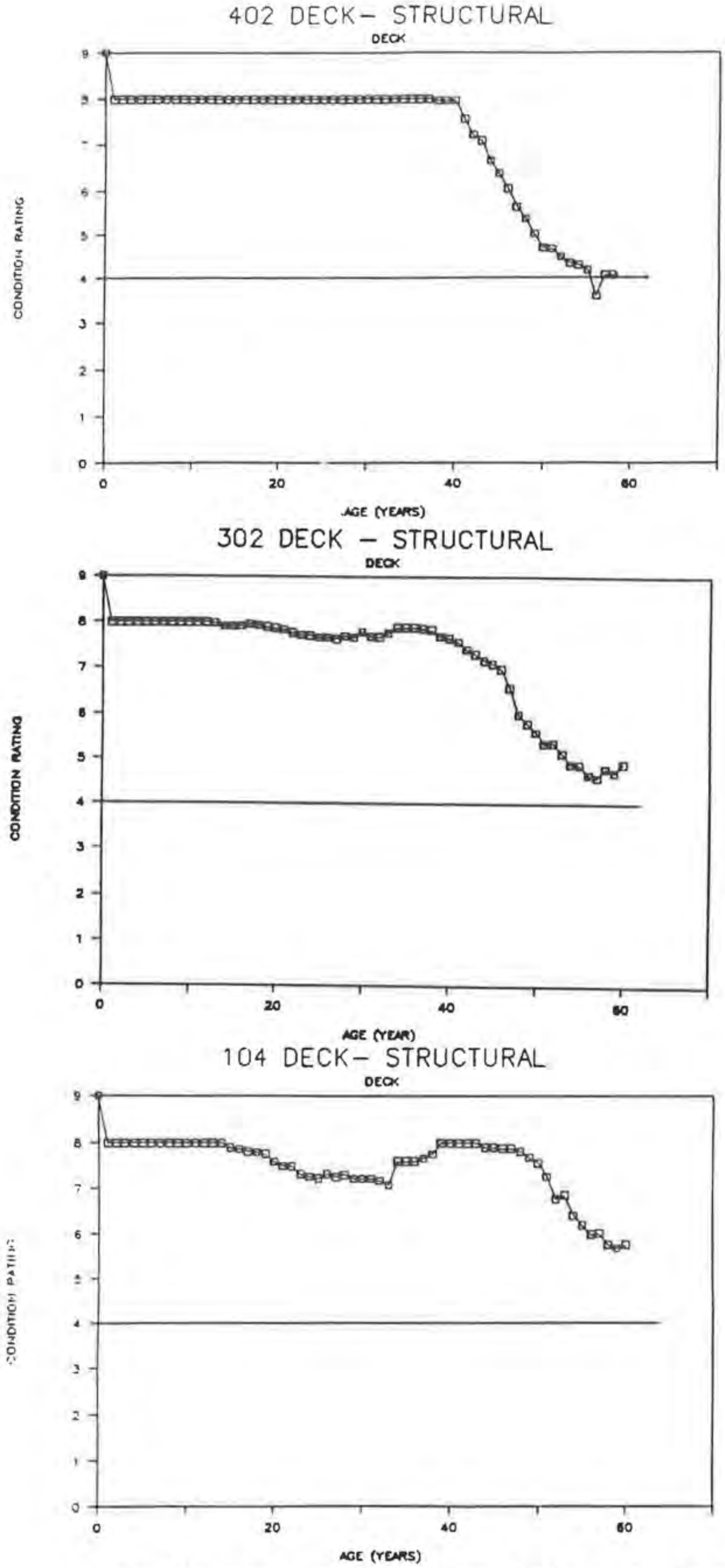


Figure 5.9 Deck Structural Condition Rating vs Age Curves for 104, 302, 402 Bridges

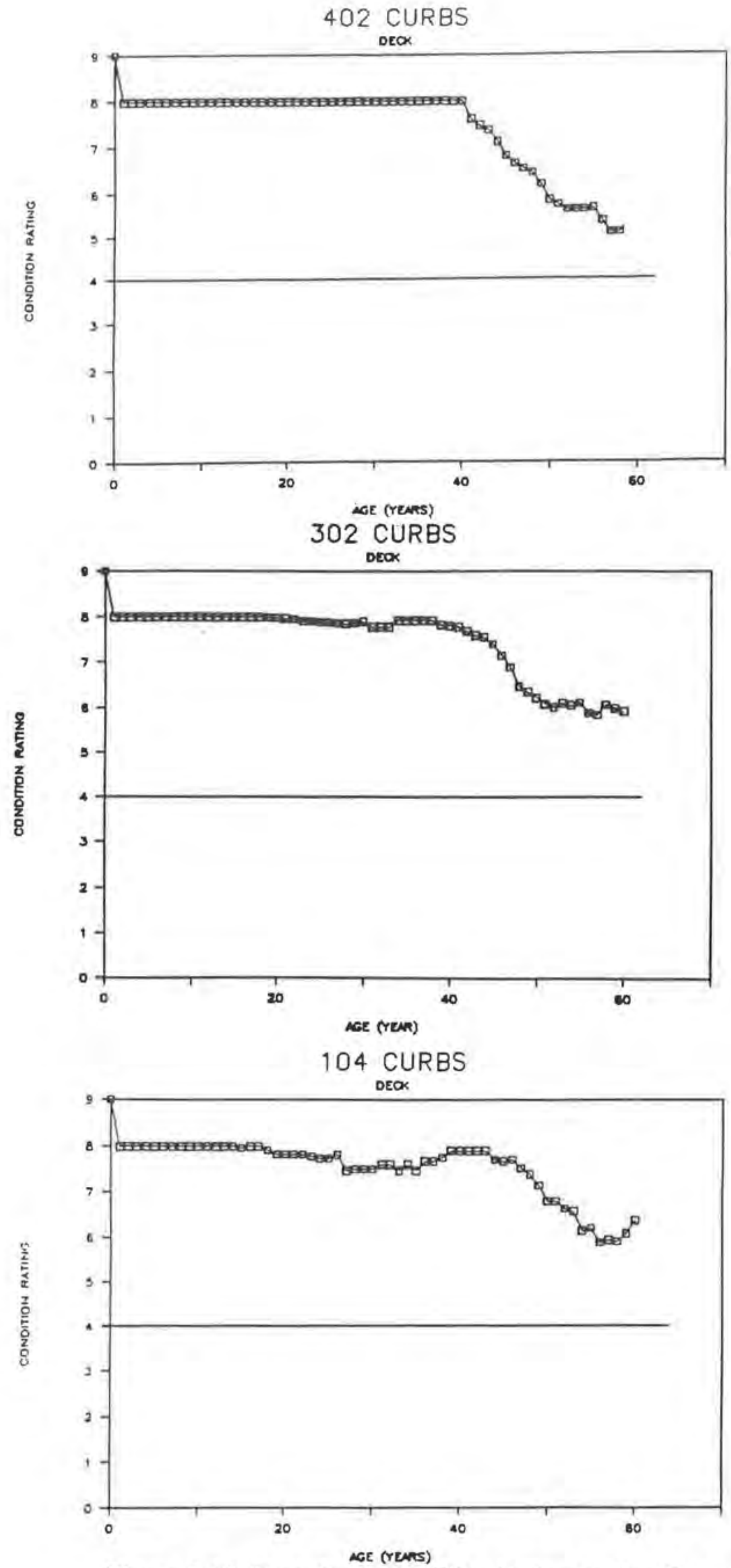
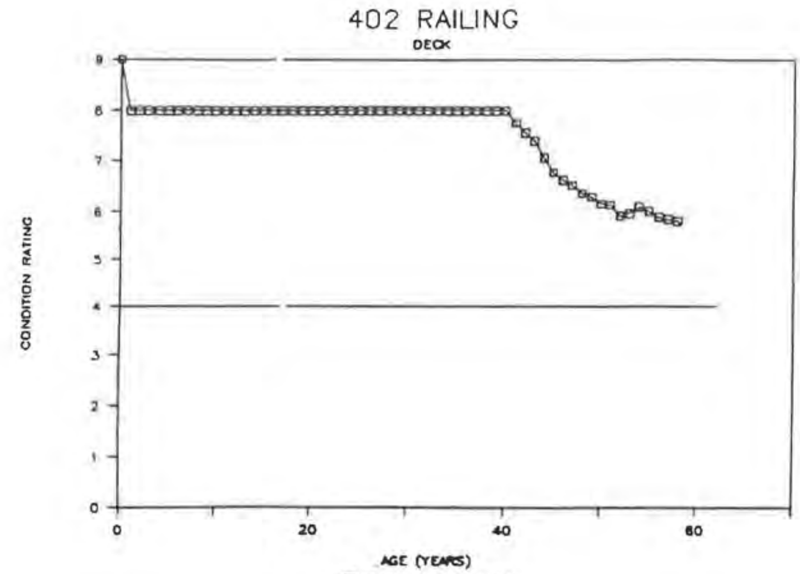
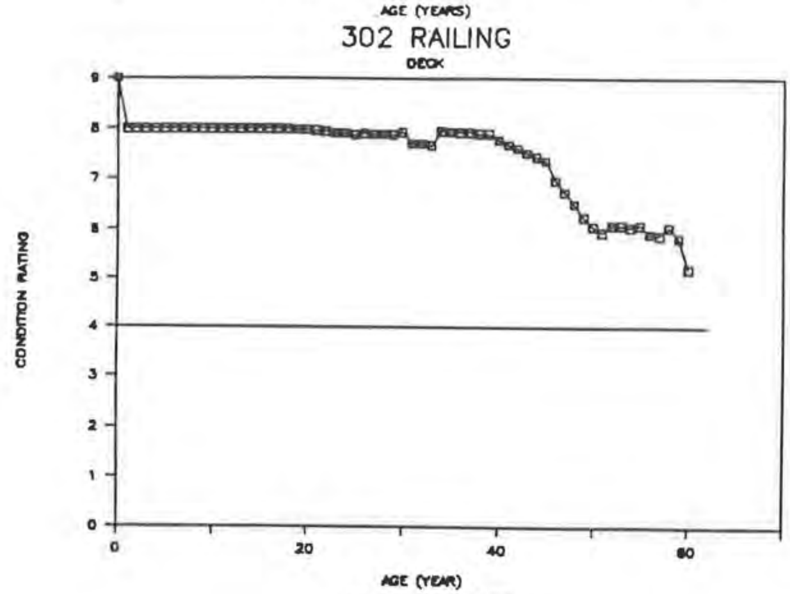


Figure 5.10 Deck Curb Condition Rating vs Age Curves for 104, 302, 402 Bridges





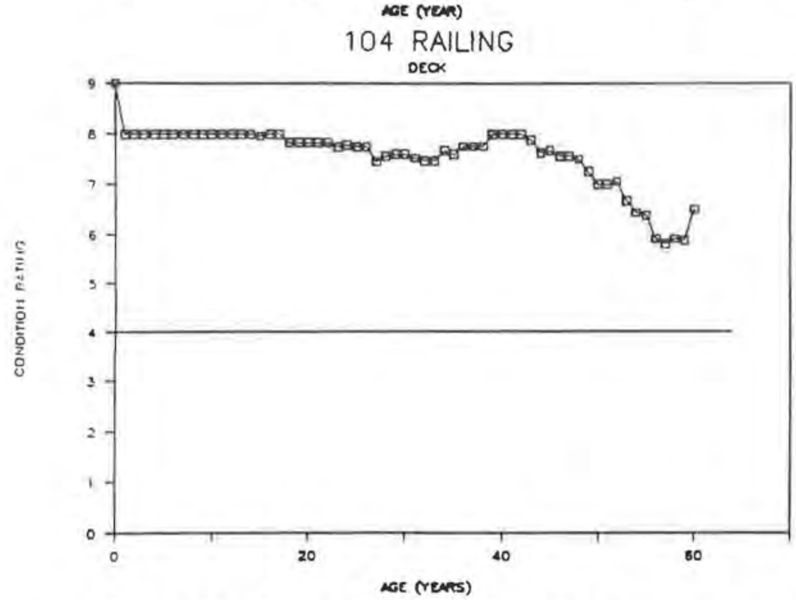


Figure 5.11 Deck Railing Condition Rating vs Age Curves for 104, 302, 402 Bridges

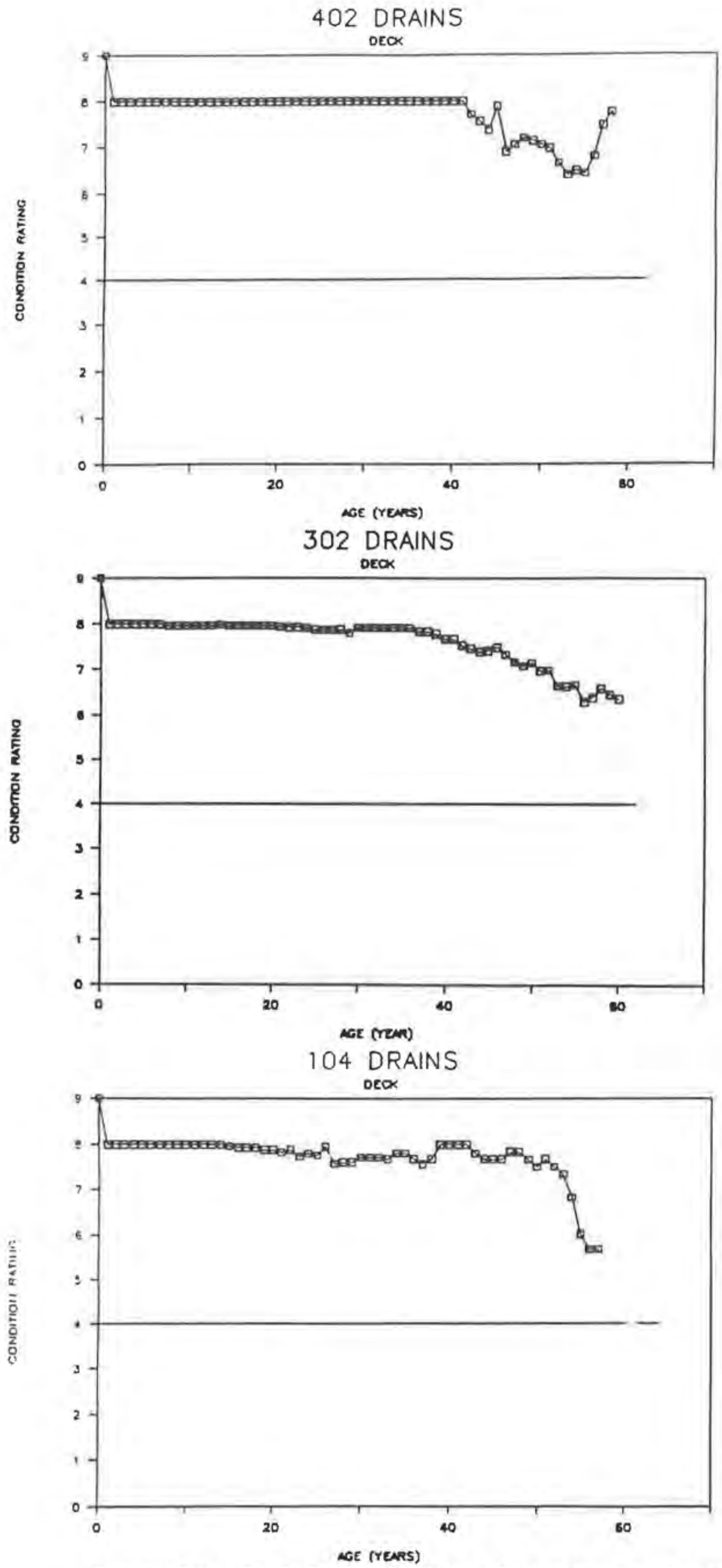
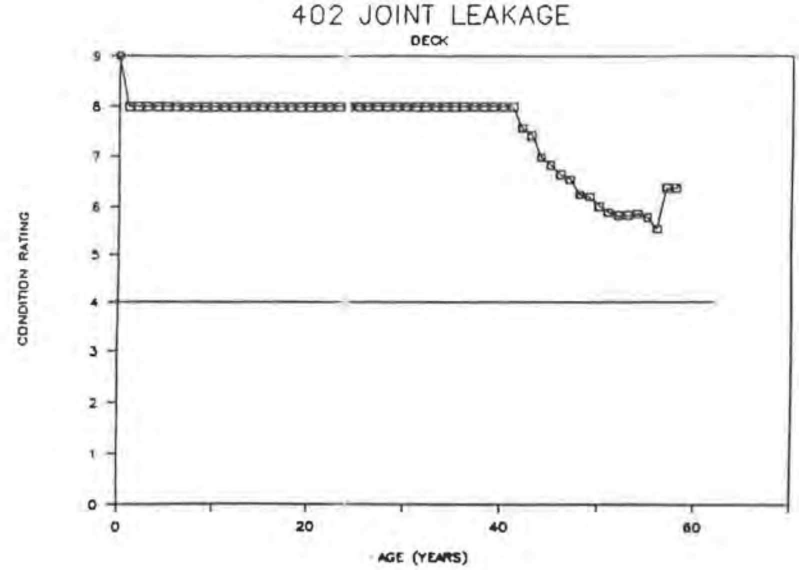


Figure 5.12 Deck Drain Condition Rating vs Age Curves for 104, 302, 402 Bridges



302 Joint Leakage
Data Not Available

104 Joint Leakage
Data Not Available

Figure 5.13 Deck Joint Leakage Condition Rating vs Age
Curves for 104, 302, 402 Bridges

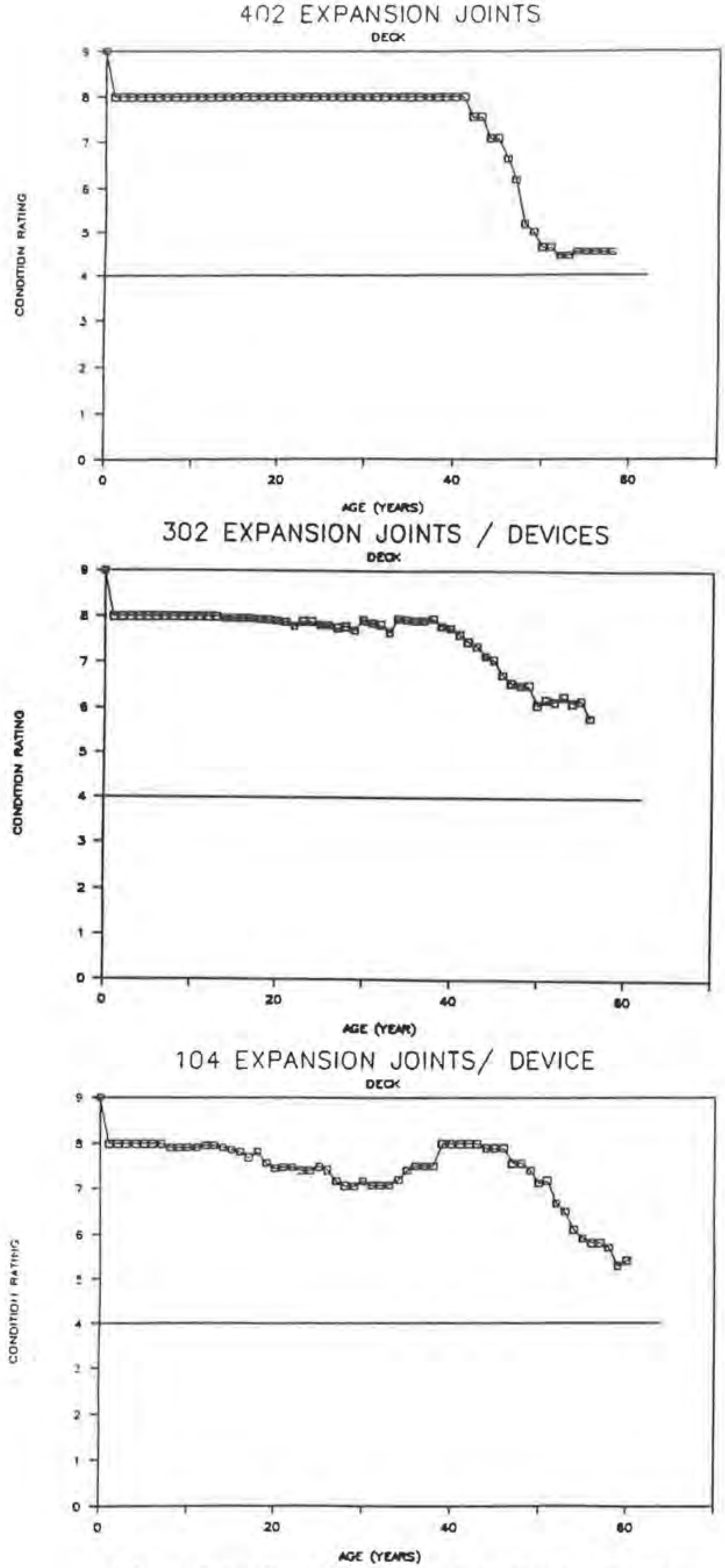


Figure 5.14 Deck Expansion Joint/Device Condition Rating vs Age Curves for 104, 302, 402 Bridges

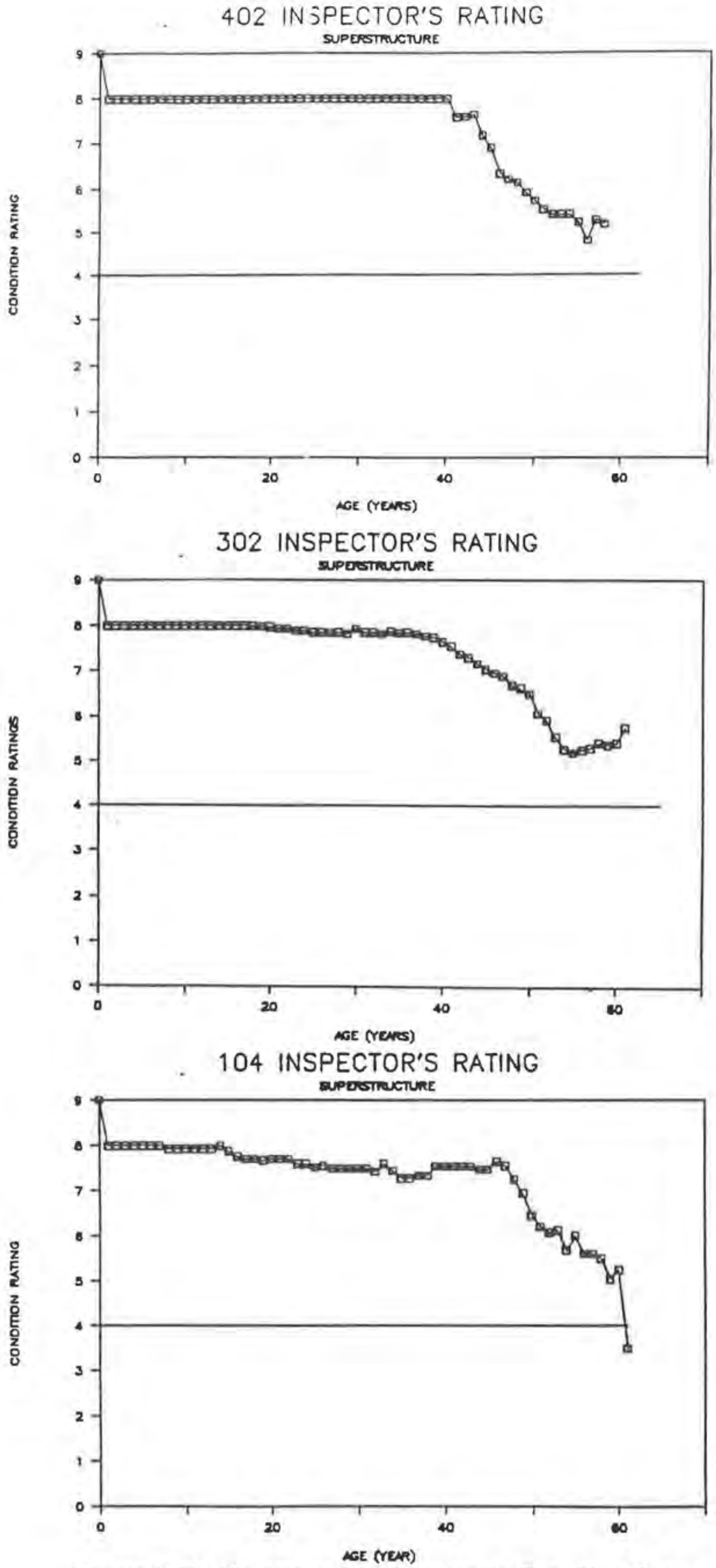


Figure 5.15 Superstructure Inspector's Condition Rating vs Age Curves for 104, 302, 402 Bridges

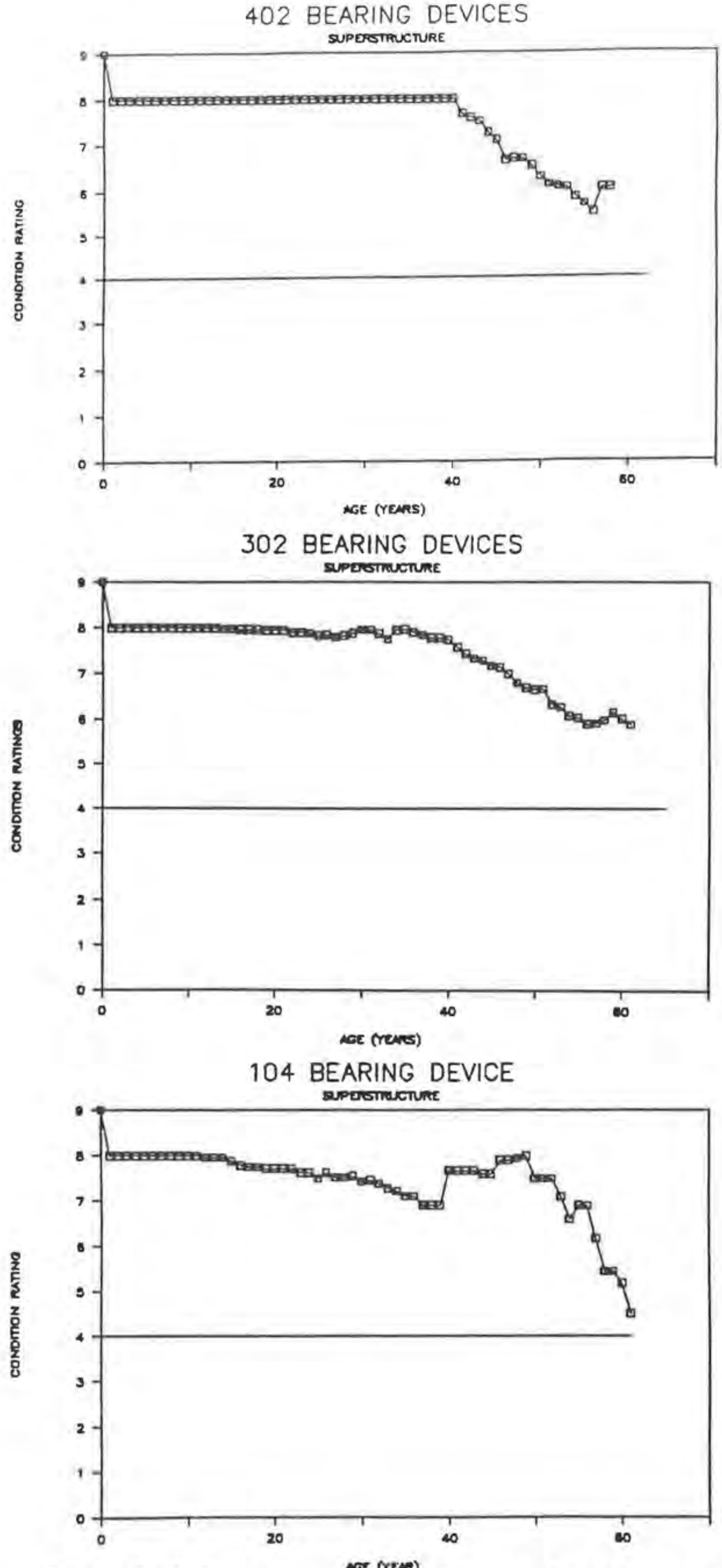


Figure 5.16 Superstructure Bearing Device Condition Rating vs Age Curves for 104, 302, 402 Bridges

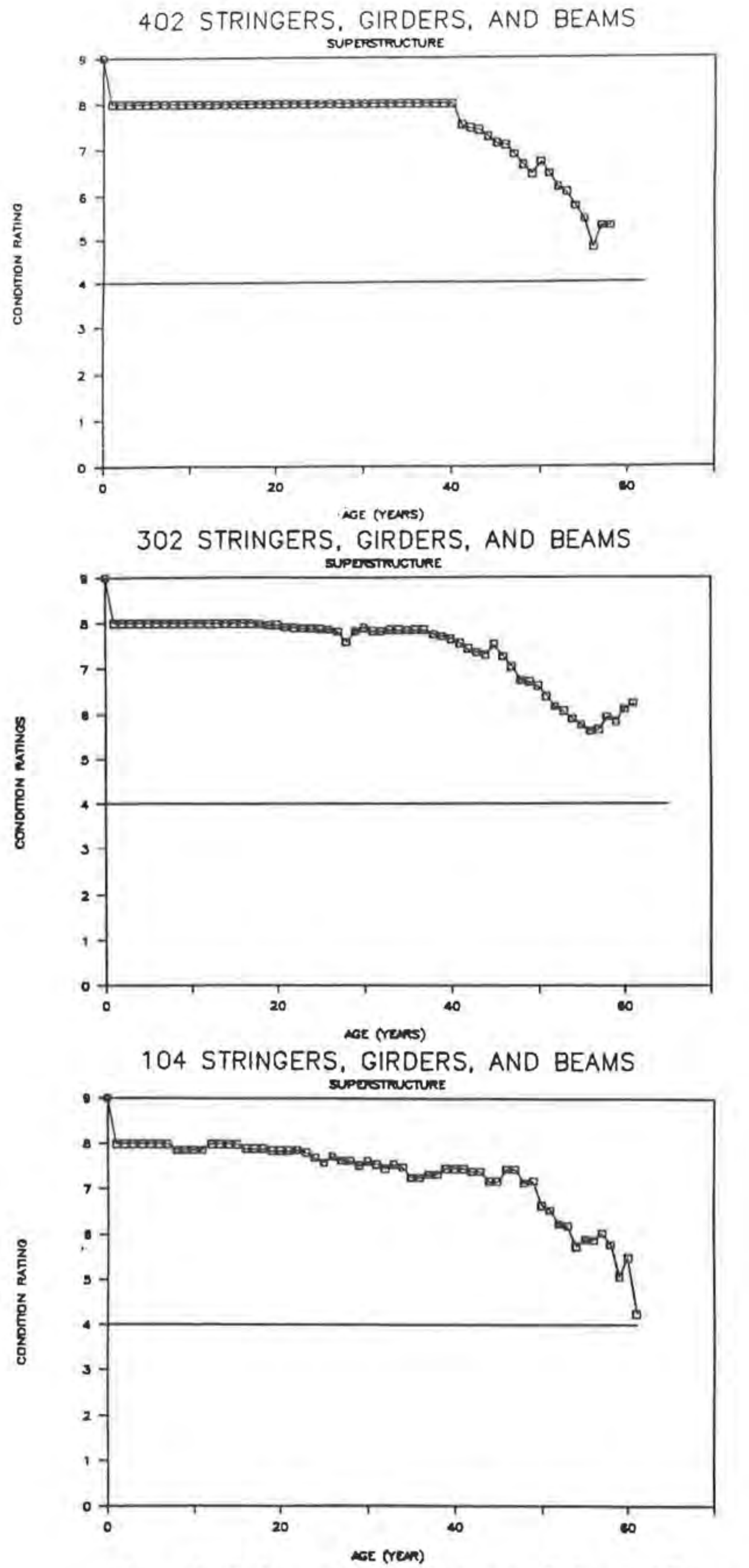


Figure 5.17 Superstructure Stringer, Girder, and Beam Condition Rating vs Age Curves for 104, 302, 402 Bridges

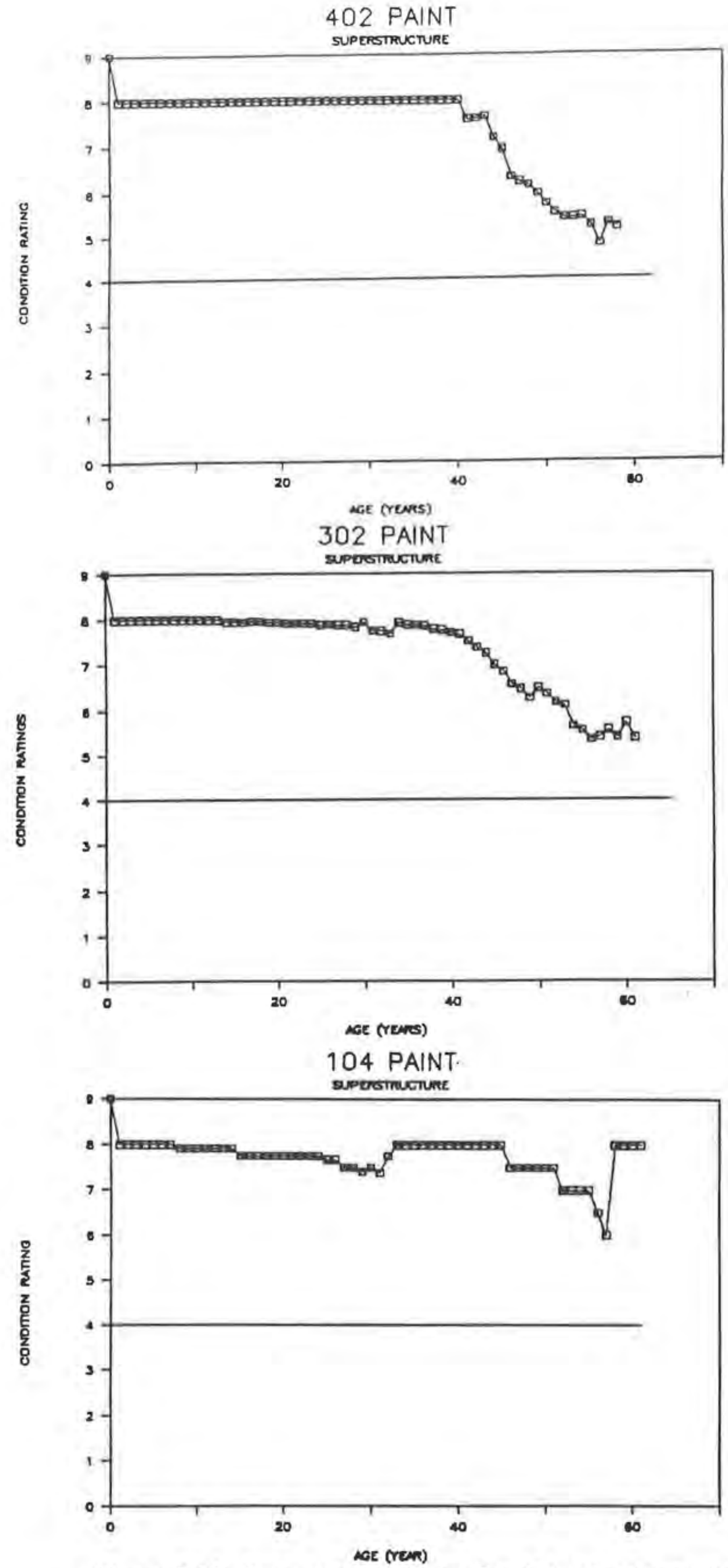


Figure 5.18 Superstructure Paint Condition Rating vs Age
Curves for 104, 302, 402 Bridges

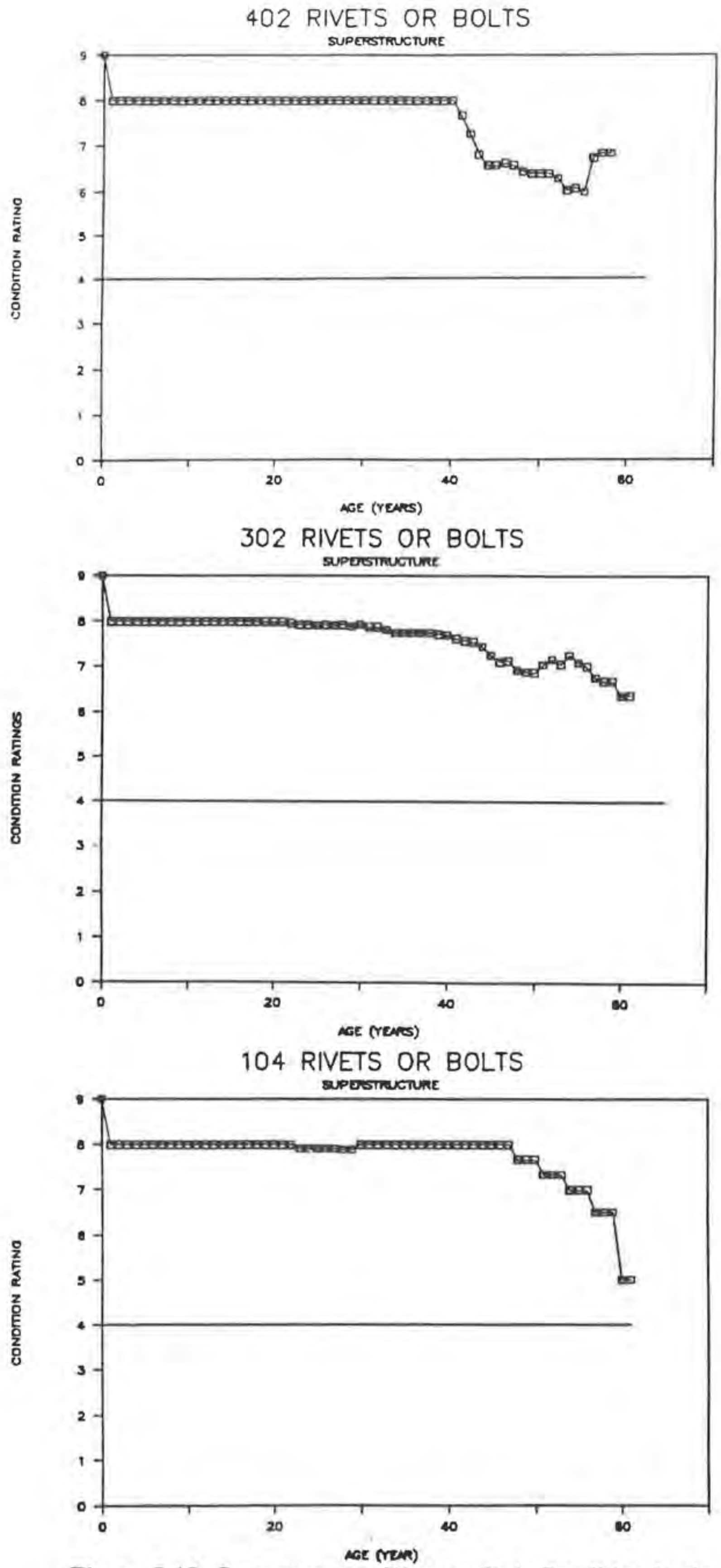
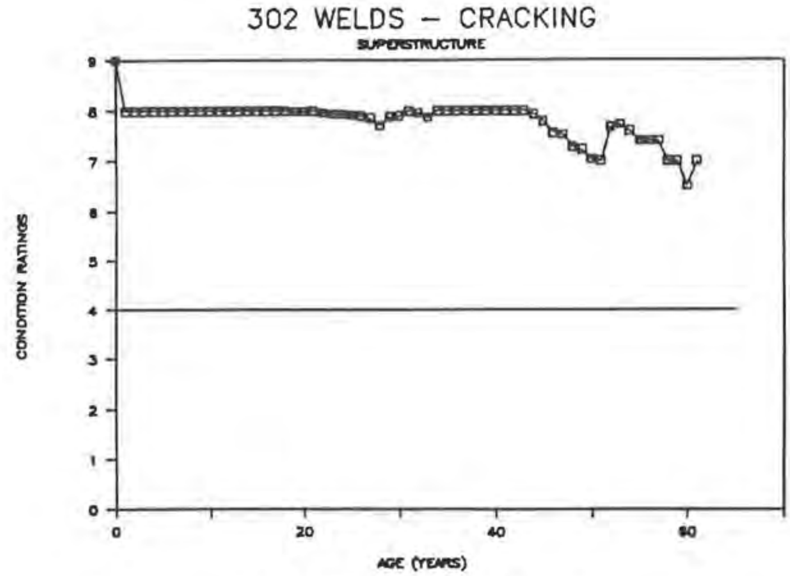


Figure 5.19 Superstructure Rivet or Bolt Condition Rating vs Age Curves for 104, 302, 402 Bridges

402 Welds-Cracking
Data Not Available



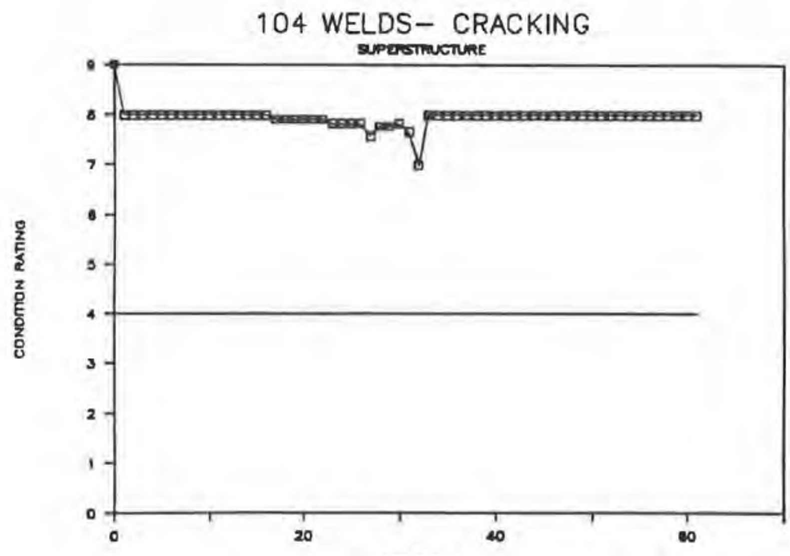


Figure 5.20 Superstructure Welds-Cracking Condition Rating vs Age Curves for 104, 302, 402 Bridges

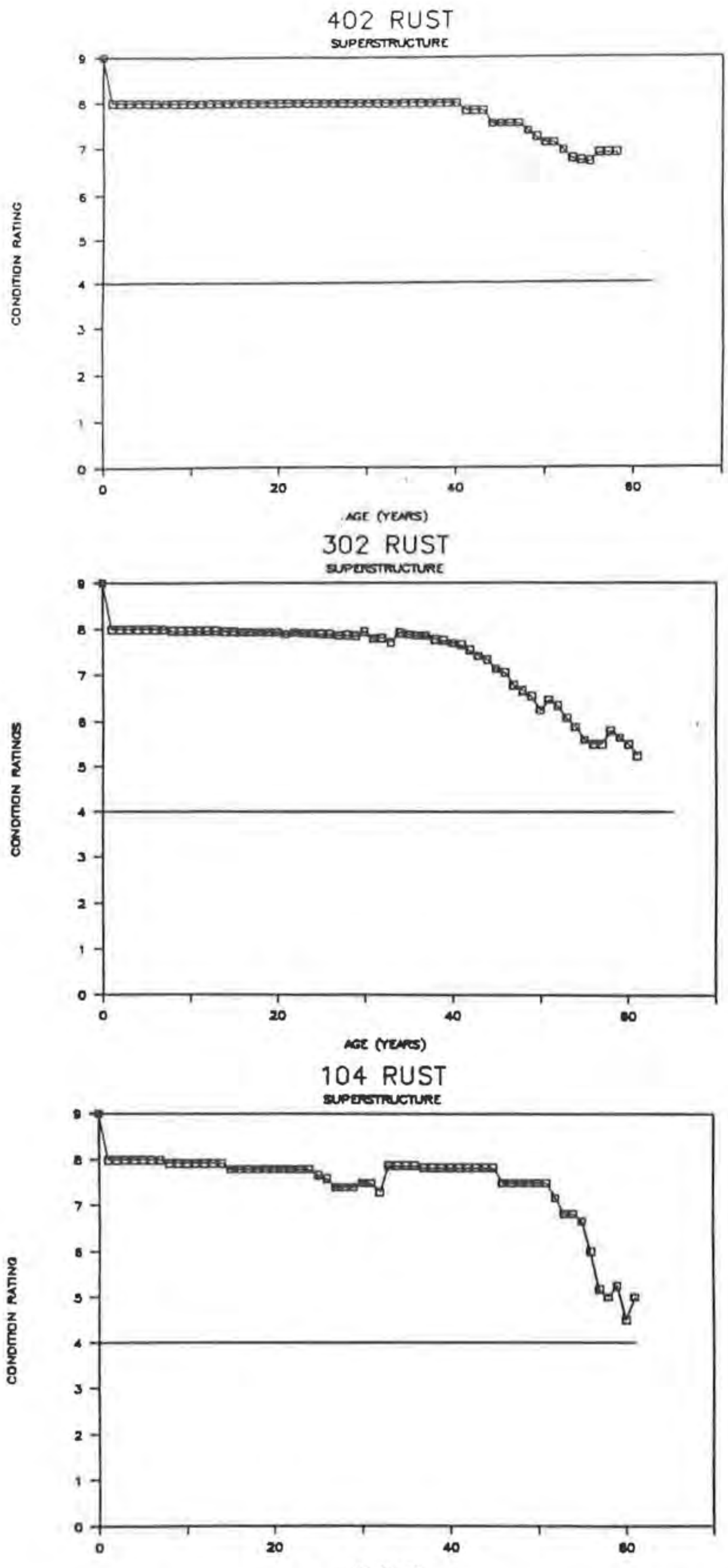
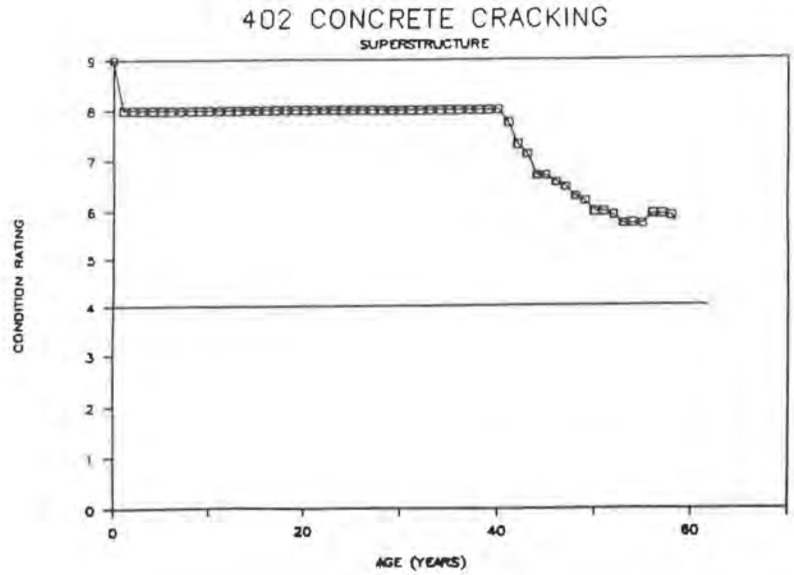


Figure 5.21 Superstructure Rust Condition Rating vs Age Curves for 104, 302, 402 Bridges



302 Concrete Cracking
Data Not Available

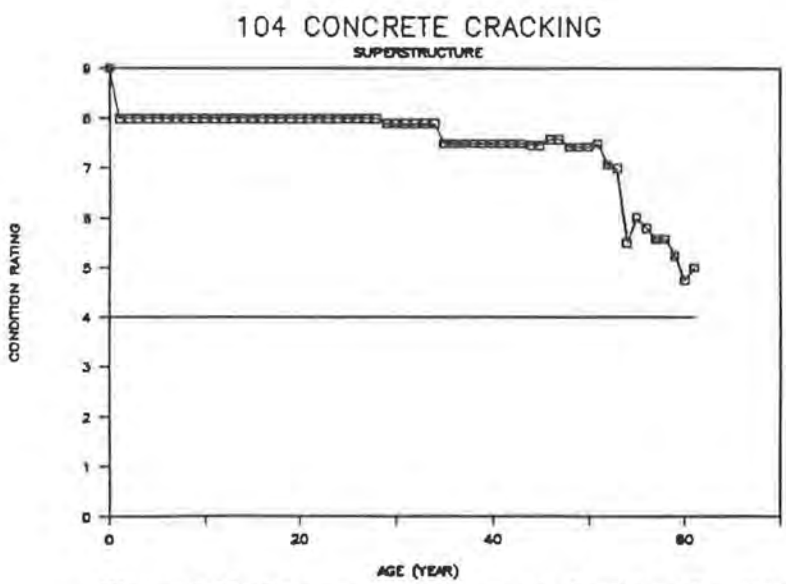


Figure 5.22 Superstructure Concrete Cracking Condition Rating vs Age Curves for 104, 302, 402 Bridges

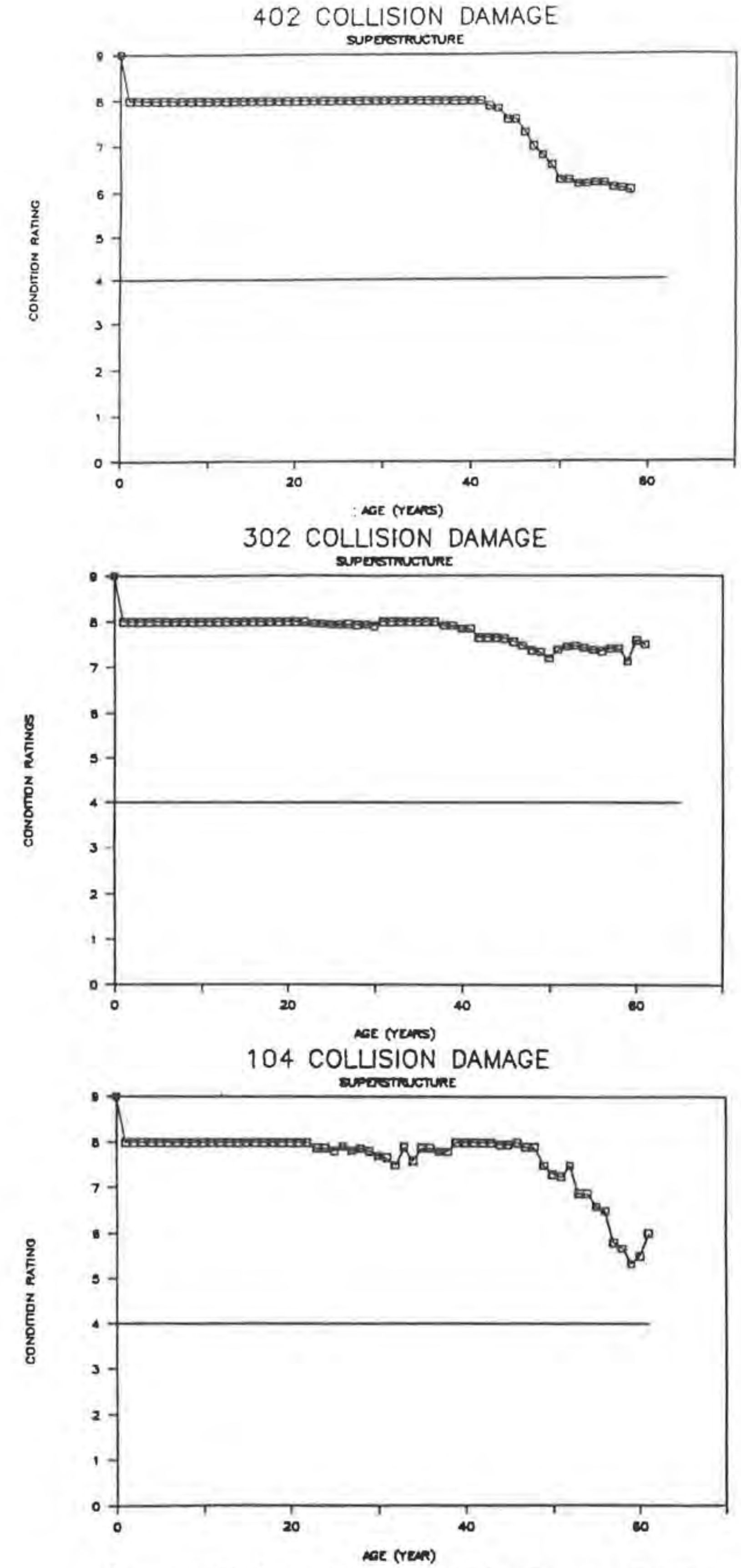
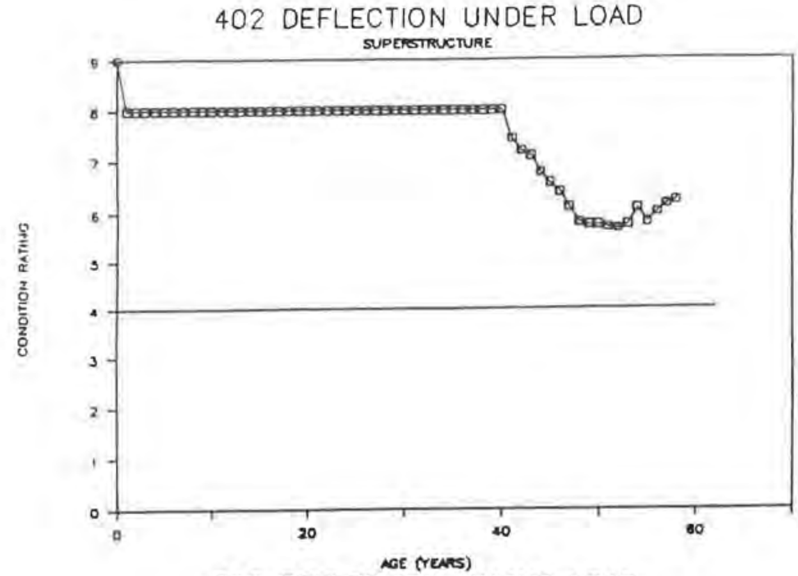
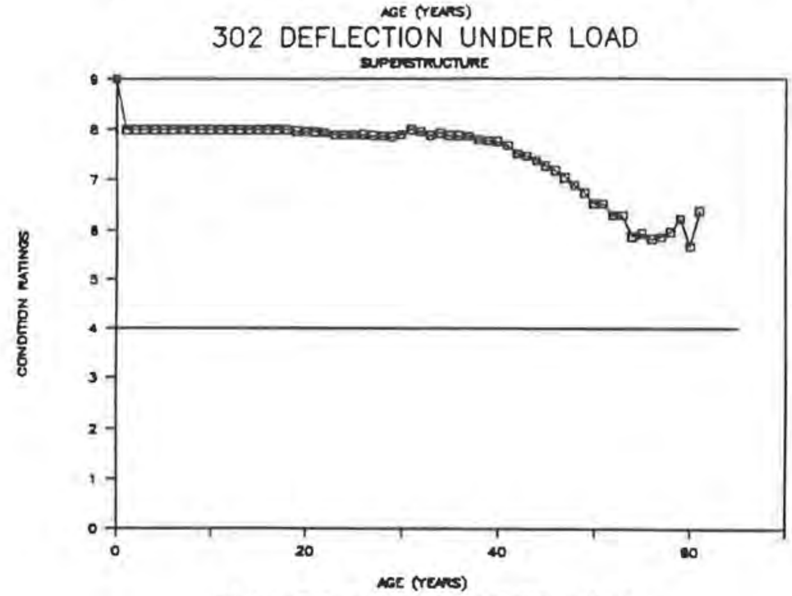


Figure 5.23 Superstructure Collision Damage Condition Rating vs Age Curves for 104, 302, 402 Bridges





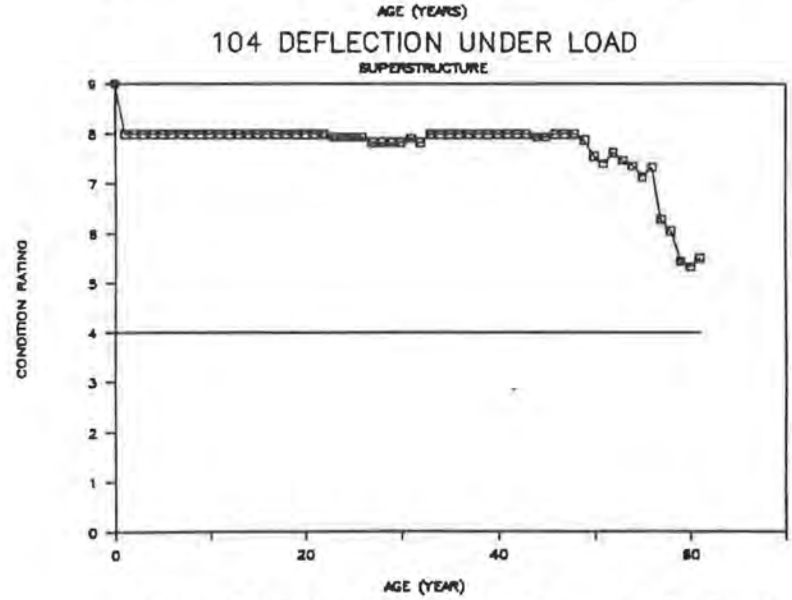
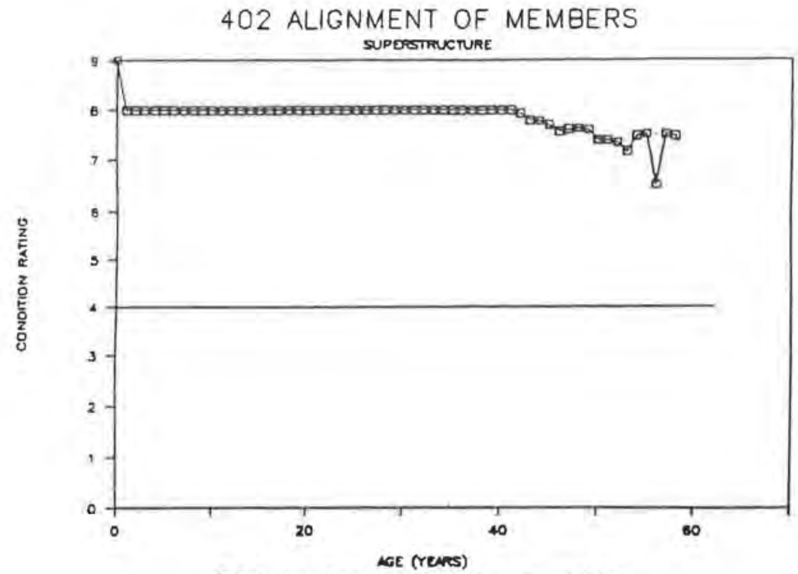
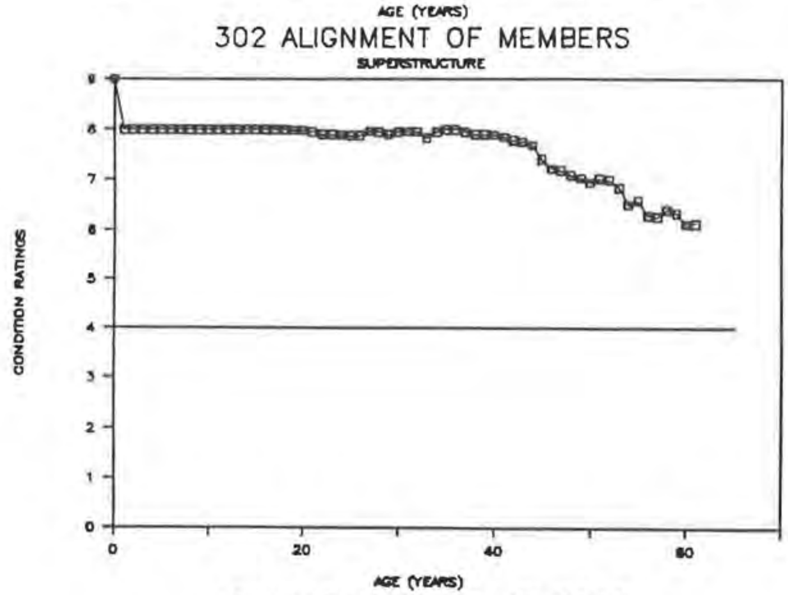


Figure 5.24 Superstructure Deflection Under Load
Condition Rating vs Age Curves for 104, 302, 402 Bridges





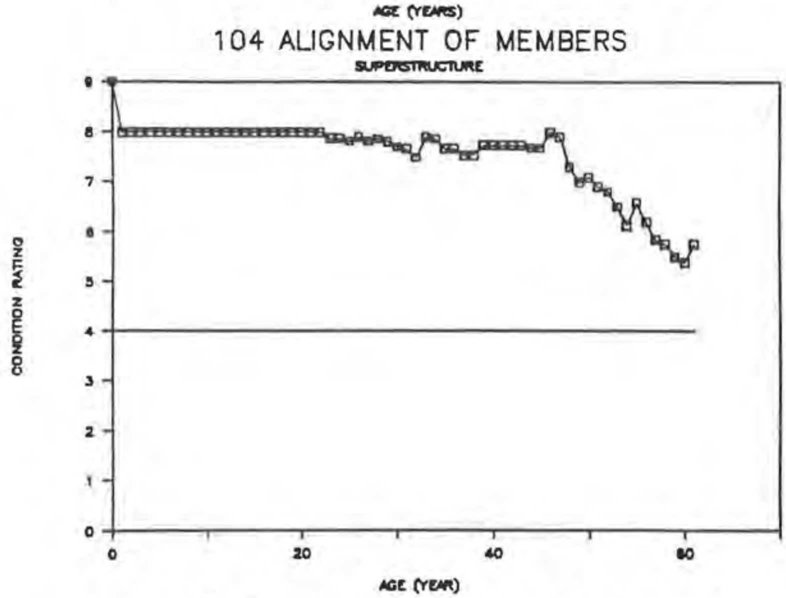


Figure 5.25 Superstructure Alignment of Member Condition Rating vs Age Curves for 104, 302, 402 Bridges

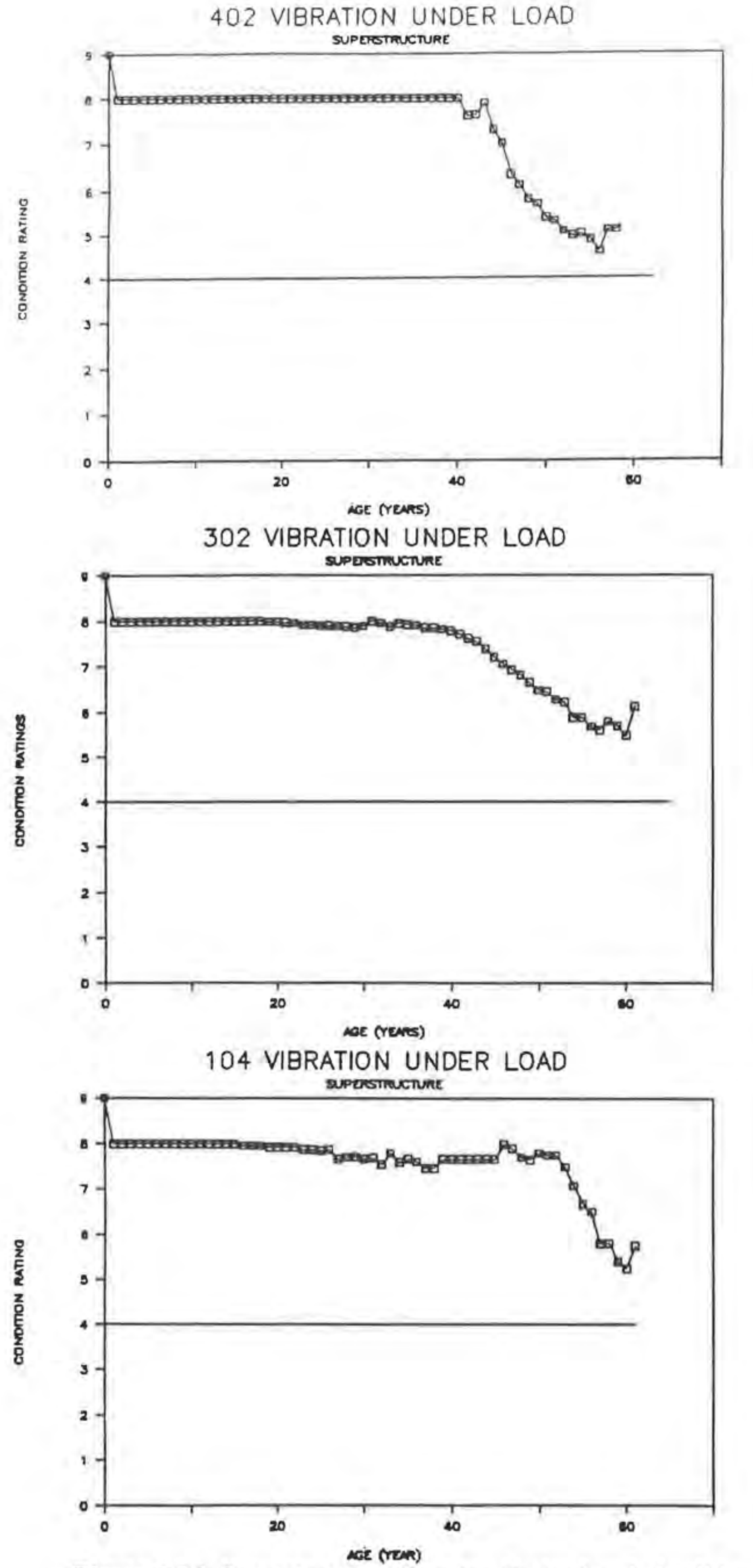


Figure 5.26 Superstructure Vibration Under Load Condition Rating vs Age Curves for 104, 302, 402 Bridges

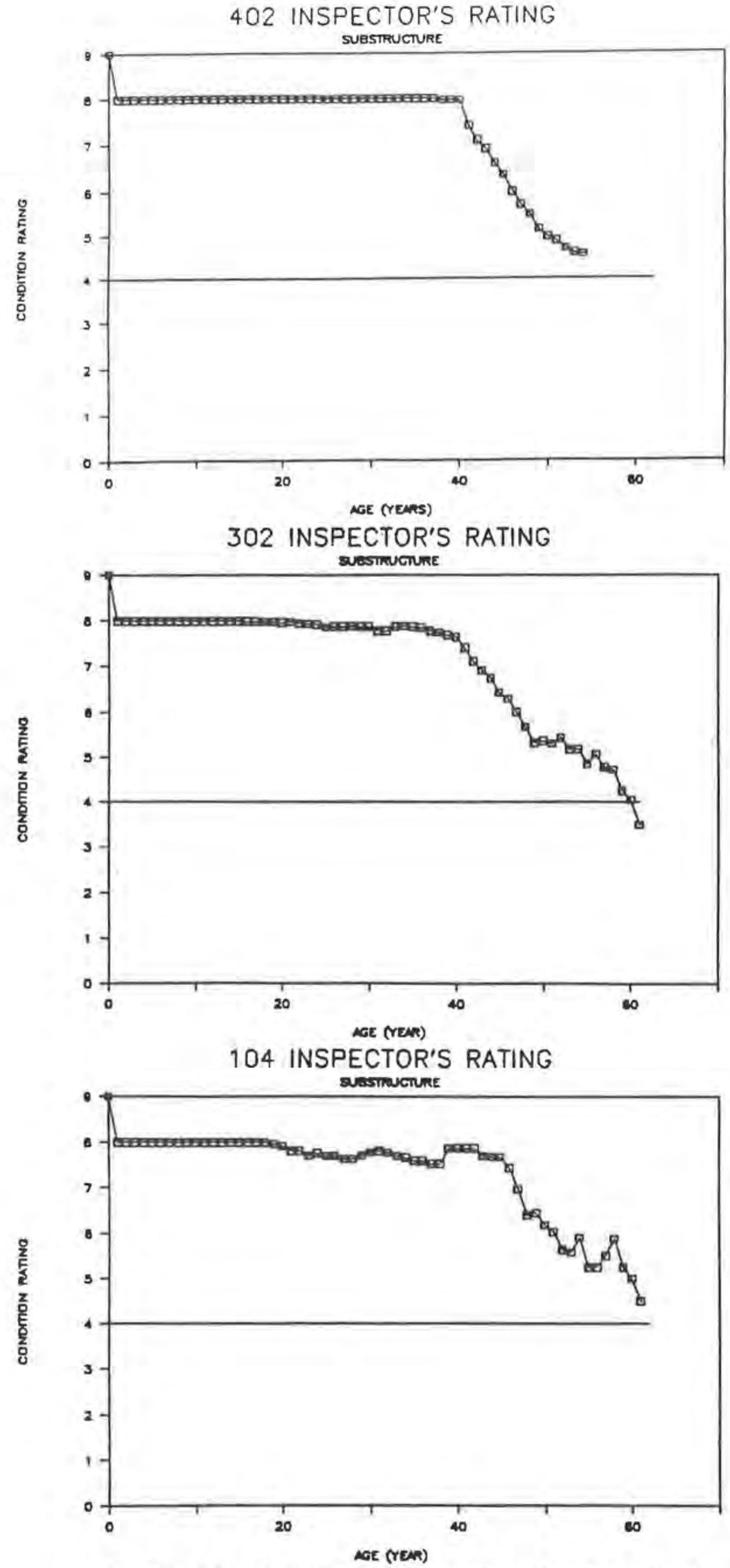


Figure 5.27 Substructure Inspector's Condition Rating vs Age Curves for 104, 302, 402 Bridges

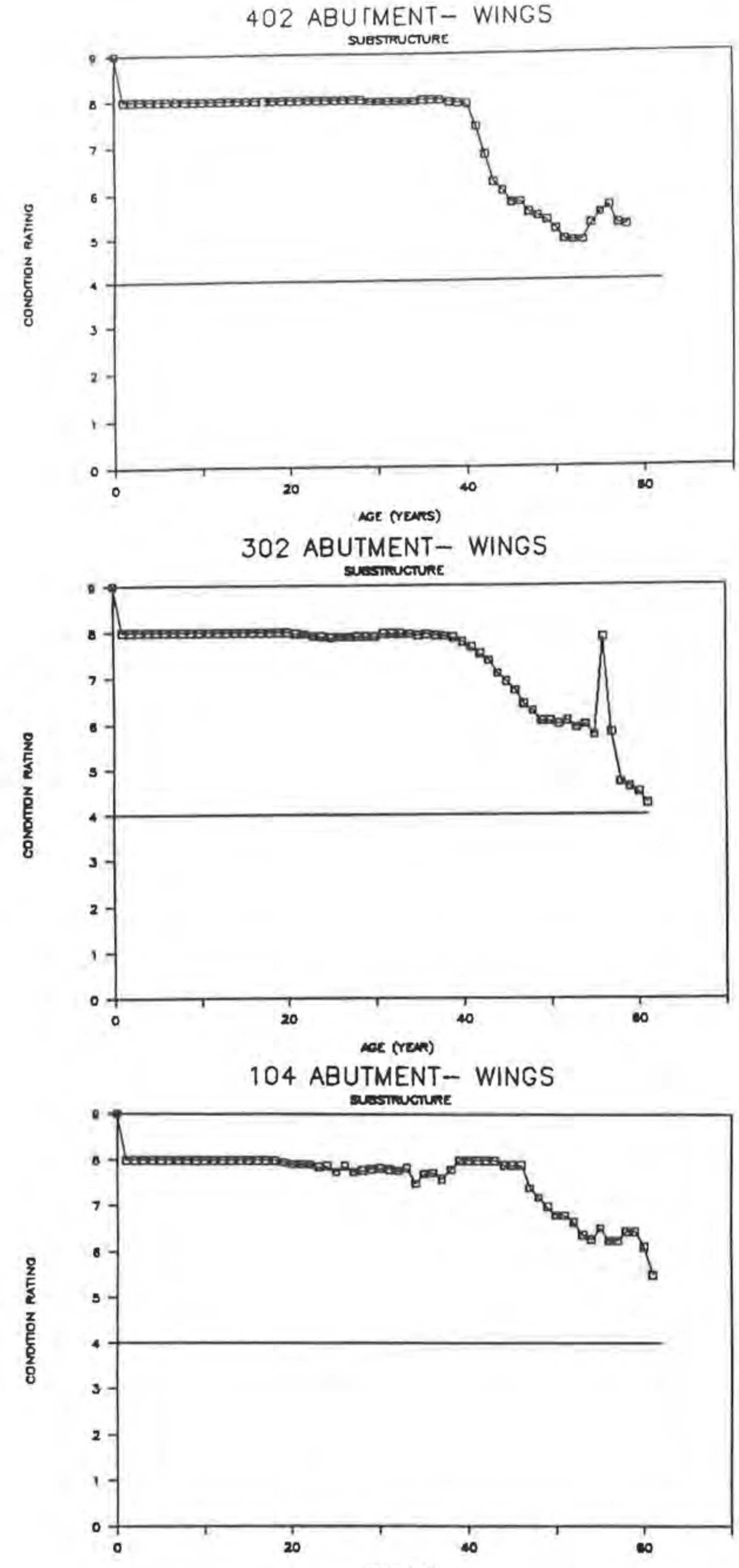


Figure 5.28 Substructure Abutment-Wing Condition Rating vs Age Curves for 104, 302, 402 Bridges

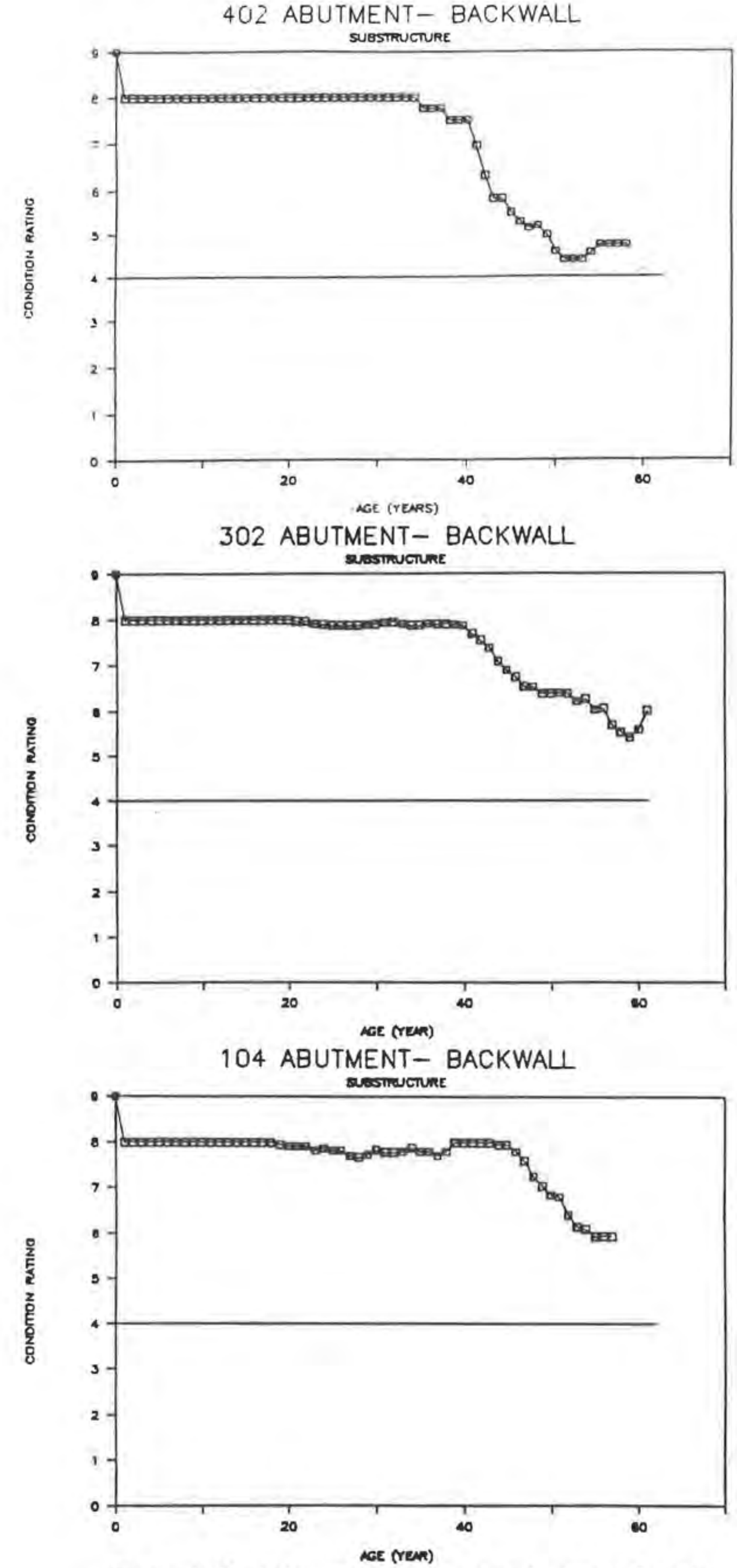


Figure 5.29 Substructure Abutment-Backwall Condition Rating vs Age Curves for 104, 302, 402 Bridges

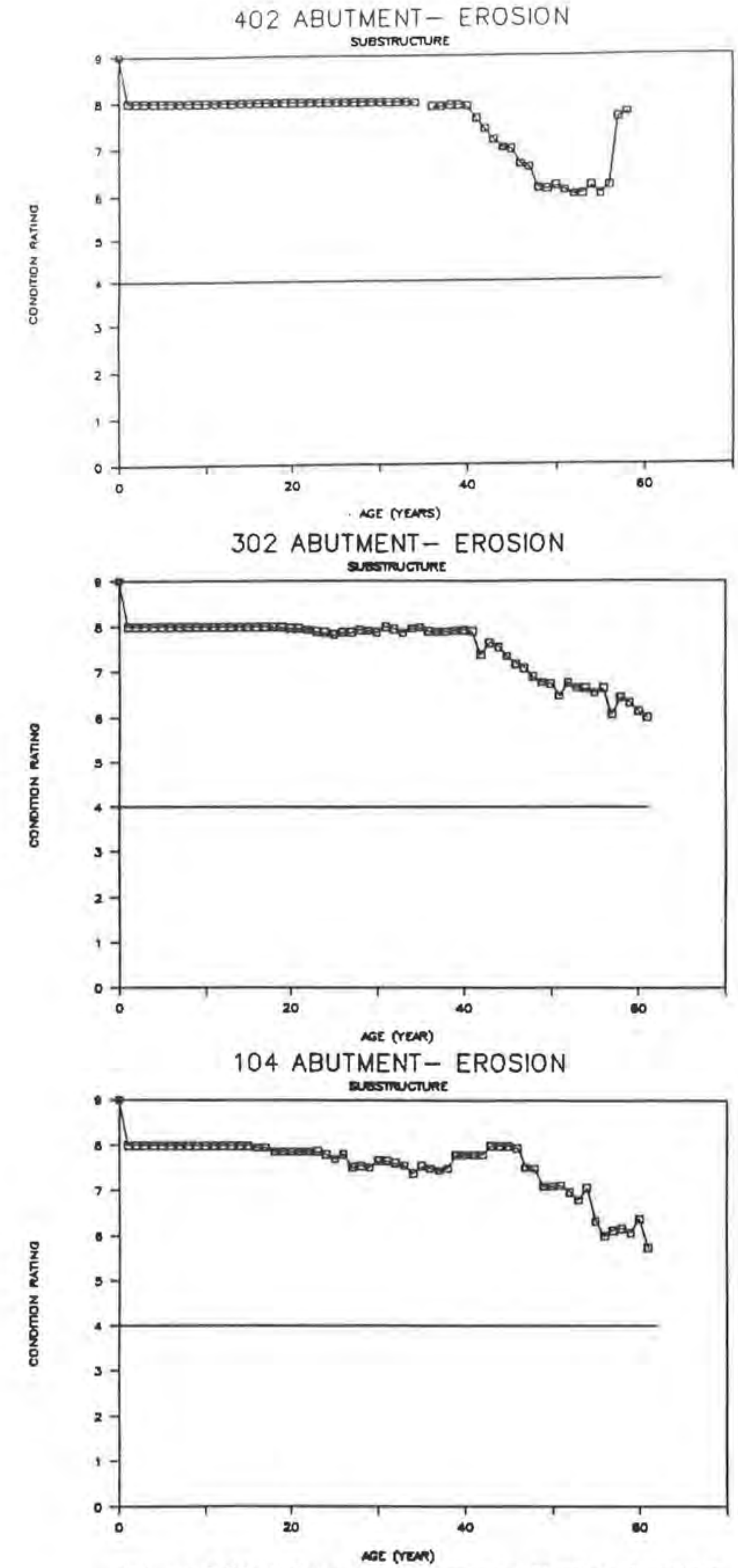
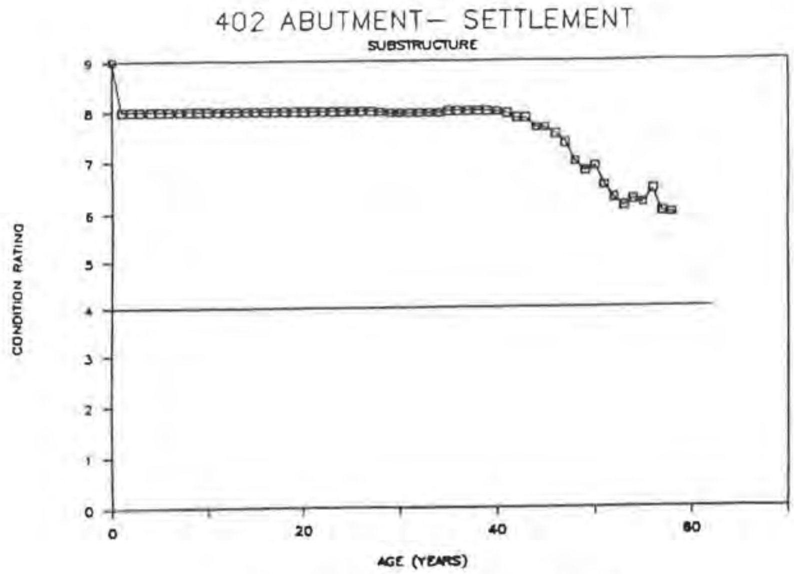
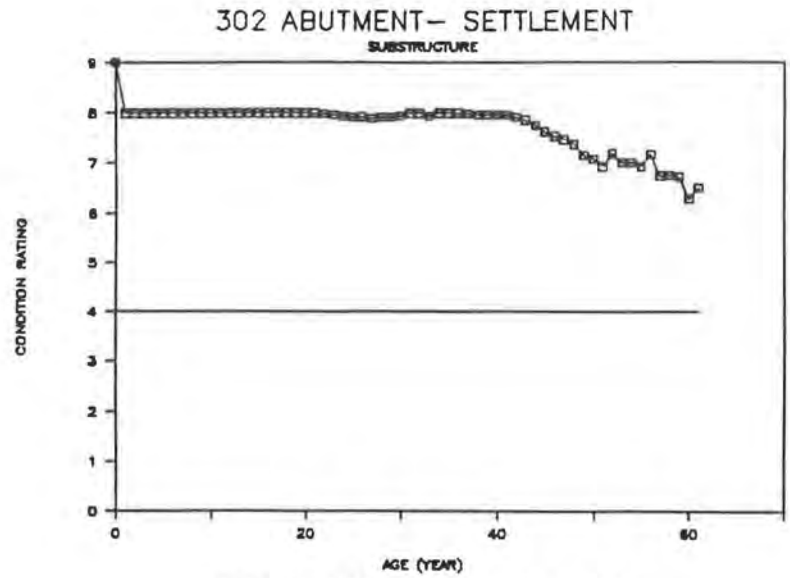


Figure 5.30 Substructure Abutment-Erosion Condition Rating vs Age Curves for 104, 302, 402 Bridges





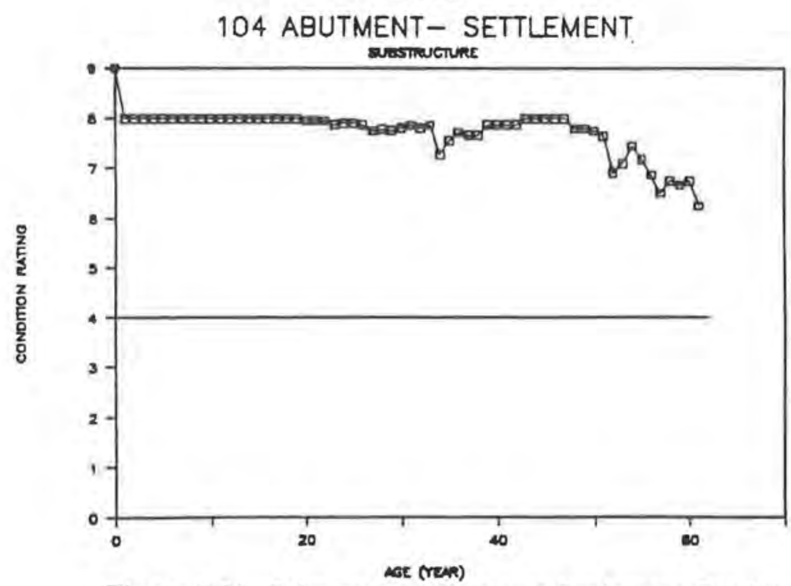


Figure 5.31 Substructure Abutment-Settlement Condition Rating vs Age Curves for 104, 302, 402 Bridges

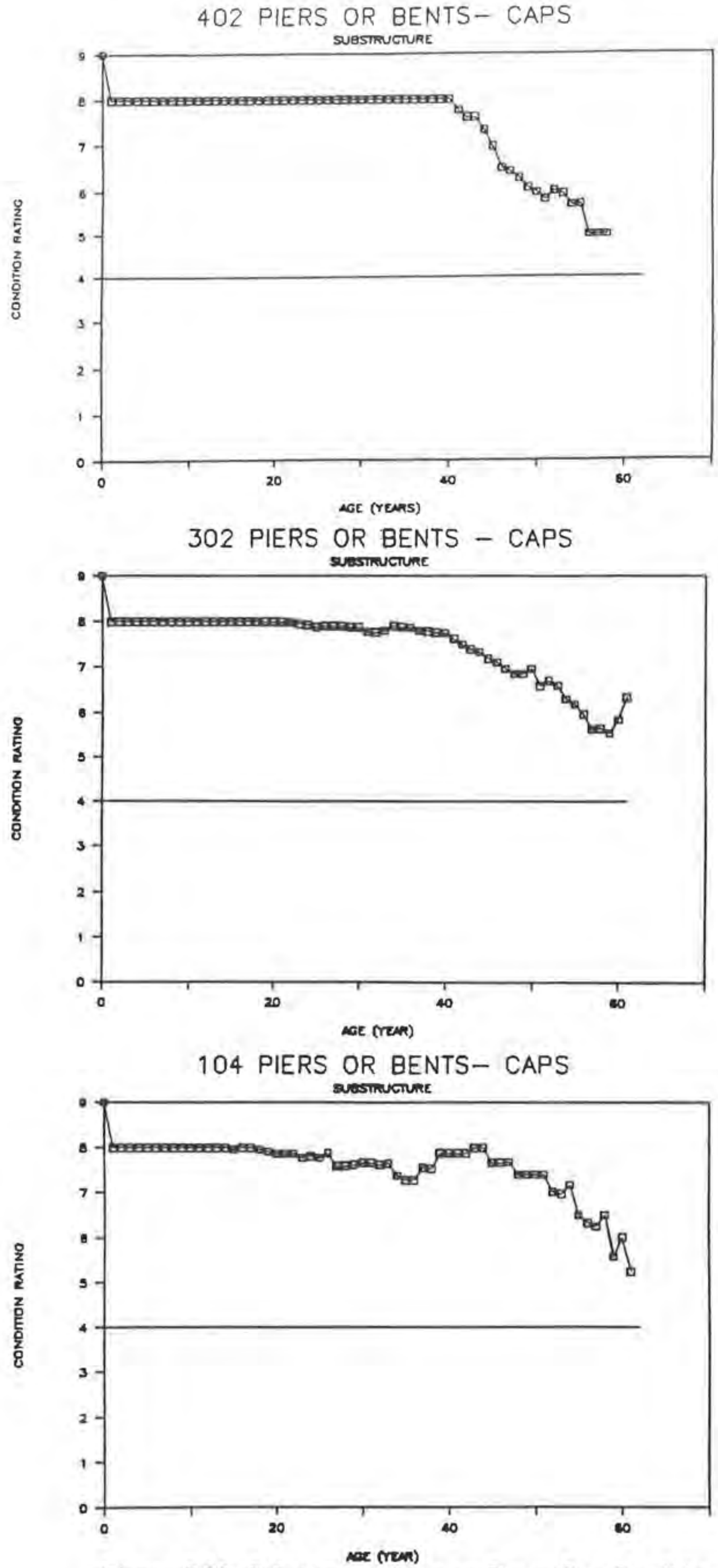


Figure 5.32 Substructure Piers or Bents-Cap Condition Rating vs Age Curves for 104, 302, 402 Bridges

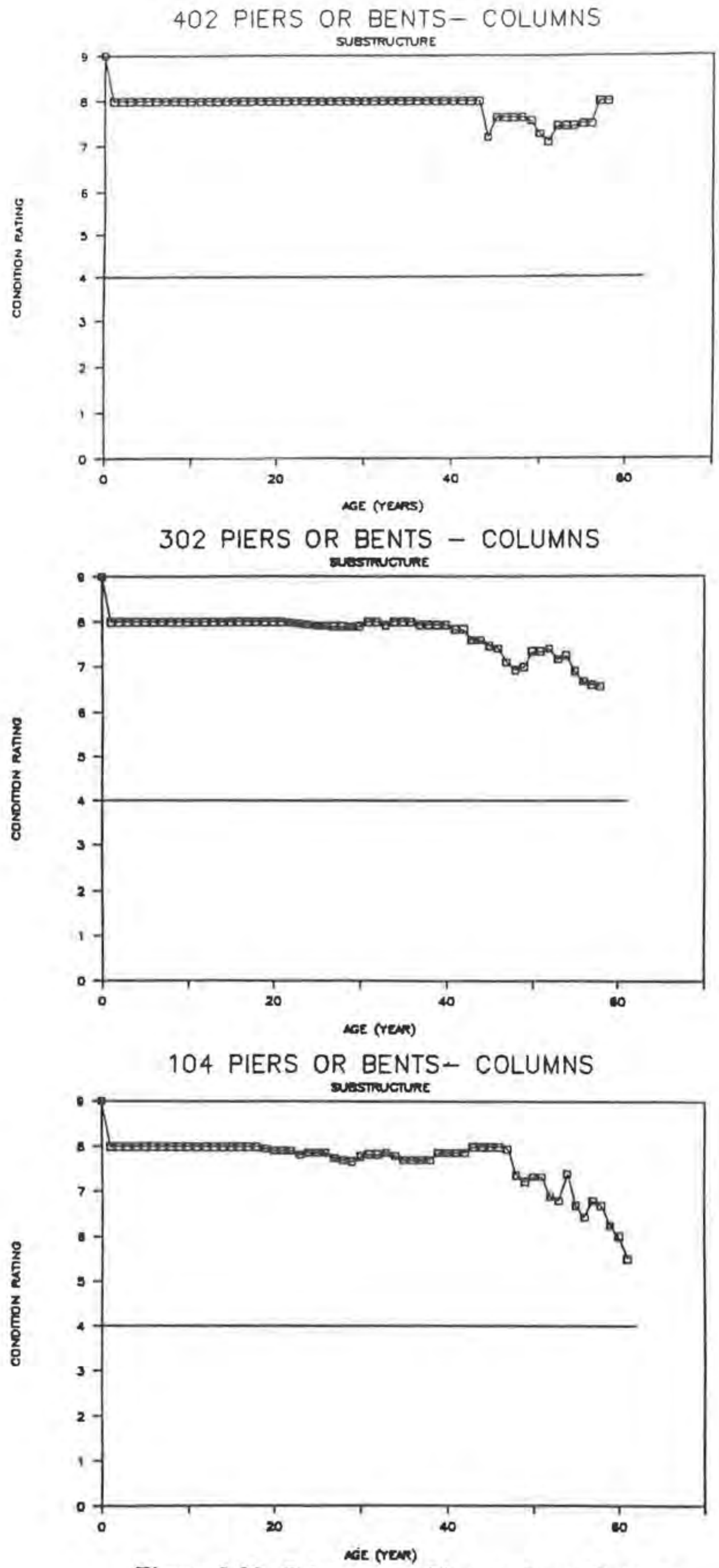


Figure 5.33 Substructure Piers or Bents-Column Condition Rating vs Age Curves for 104, 302, 402 Bridges

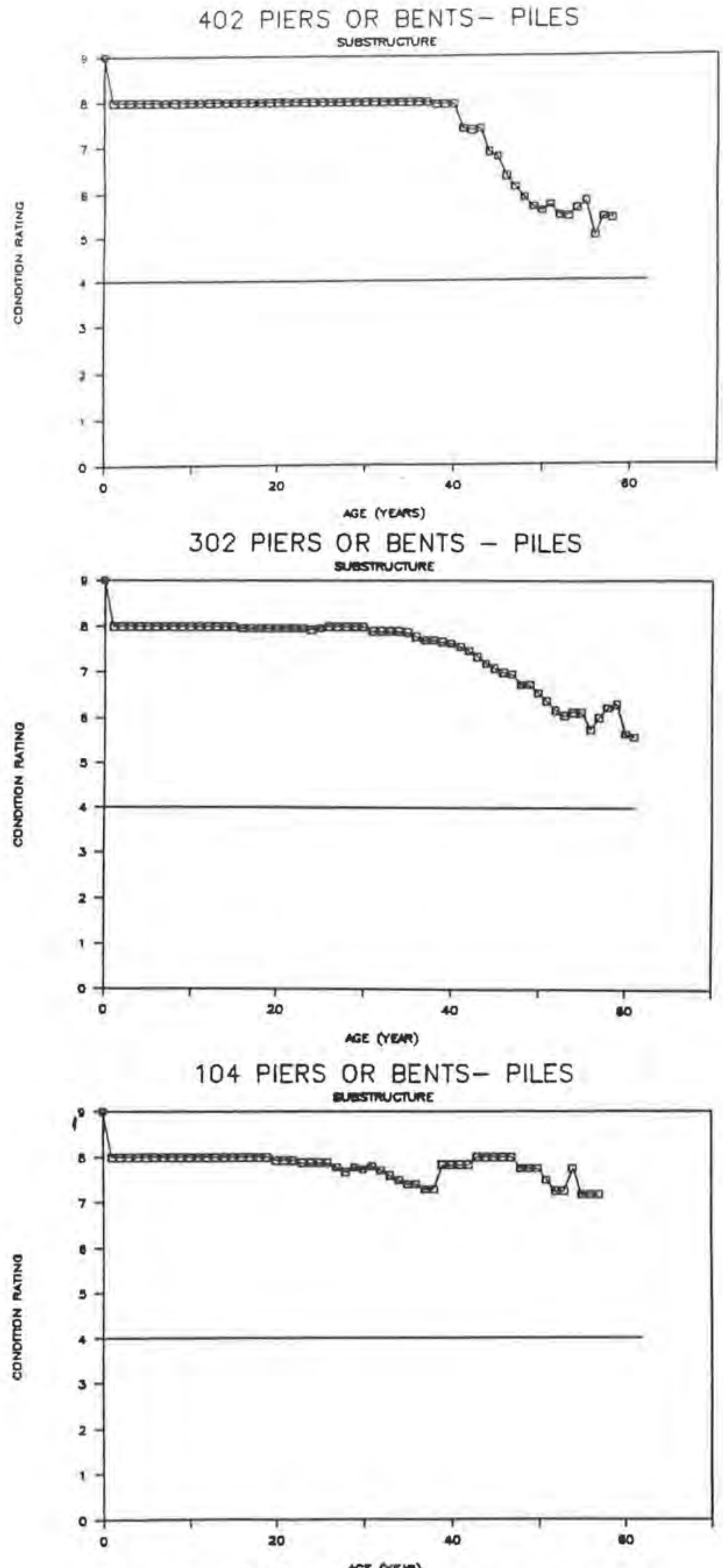
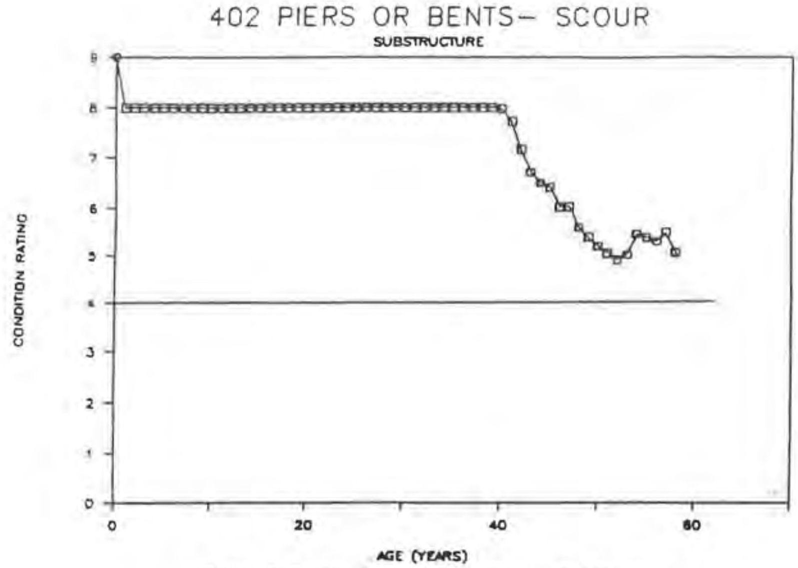
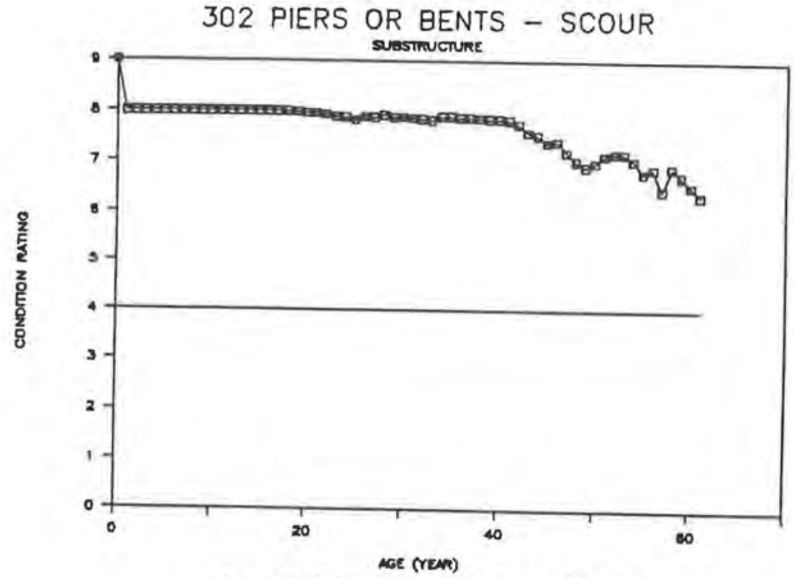


Figure 5.34 Substructure Piers or Bents-Pile
Condition Rating vs Age Curves for 104, 302, 402 Bridges





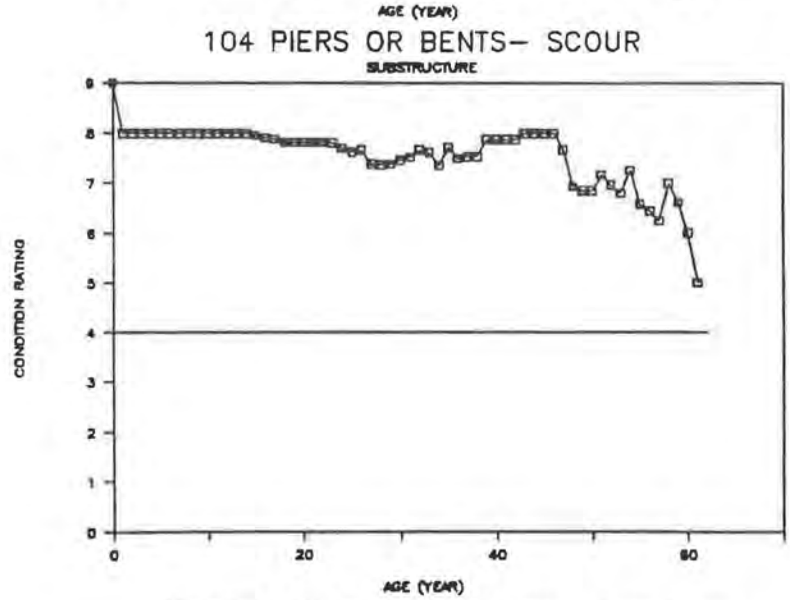


Figure 5.35 Substructure Piers or Bents-Scour
Condition Rating vs Age Curves for 104, 302, 402 Bridges

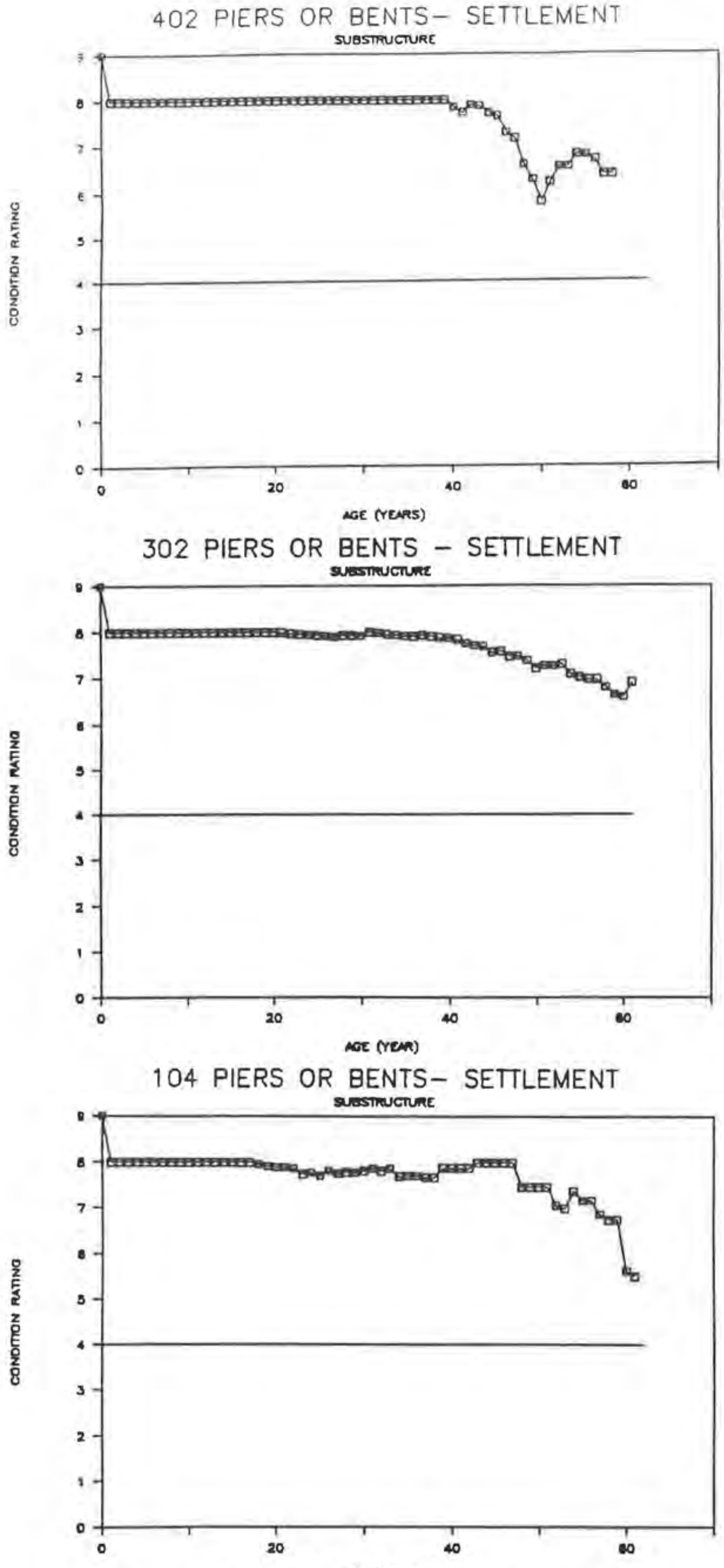
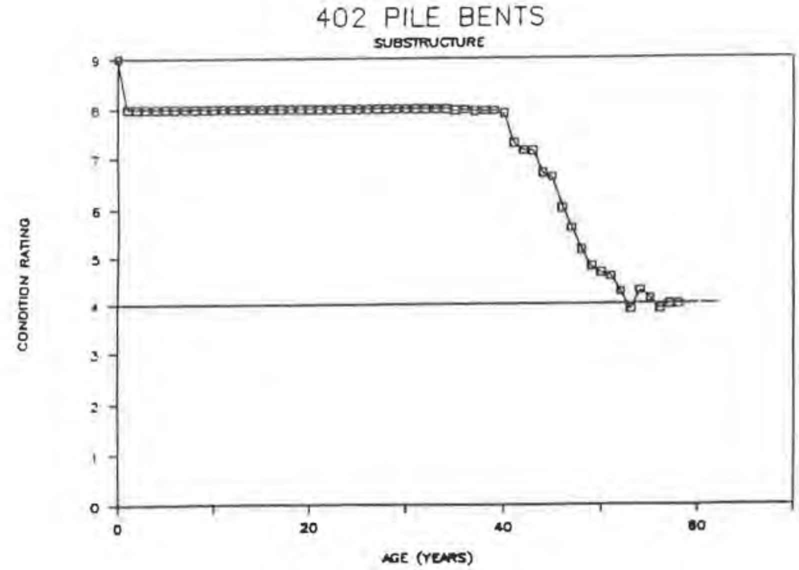
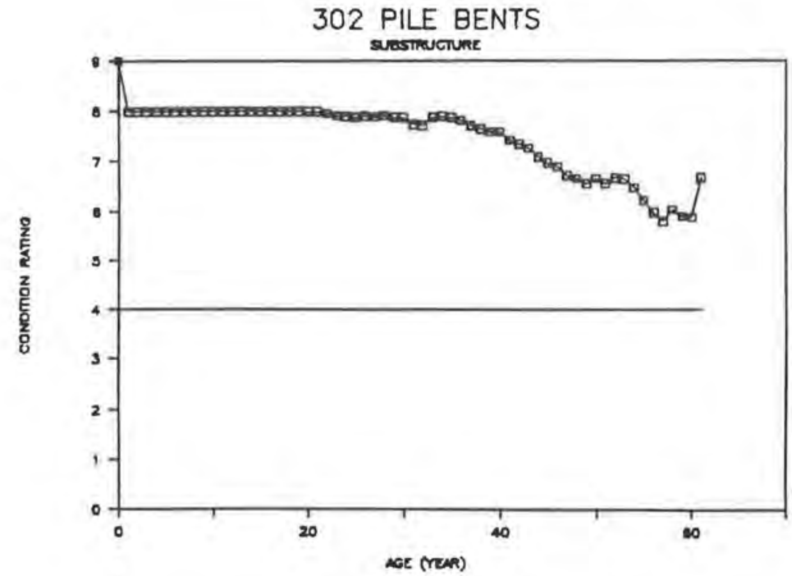


Figure 5.36 Substructure Piers or Bents-Settlement
Condition Rating vs Age Curves for 104, 302, 402 Bridges





104 Pile Bents
Data Not Available

Figure 5.37 Substructure Pile Bent Condition Rating vs Age Curves for 104, 302, 402 Bridges

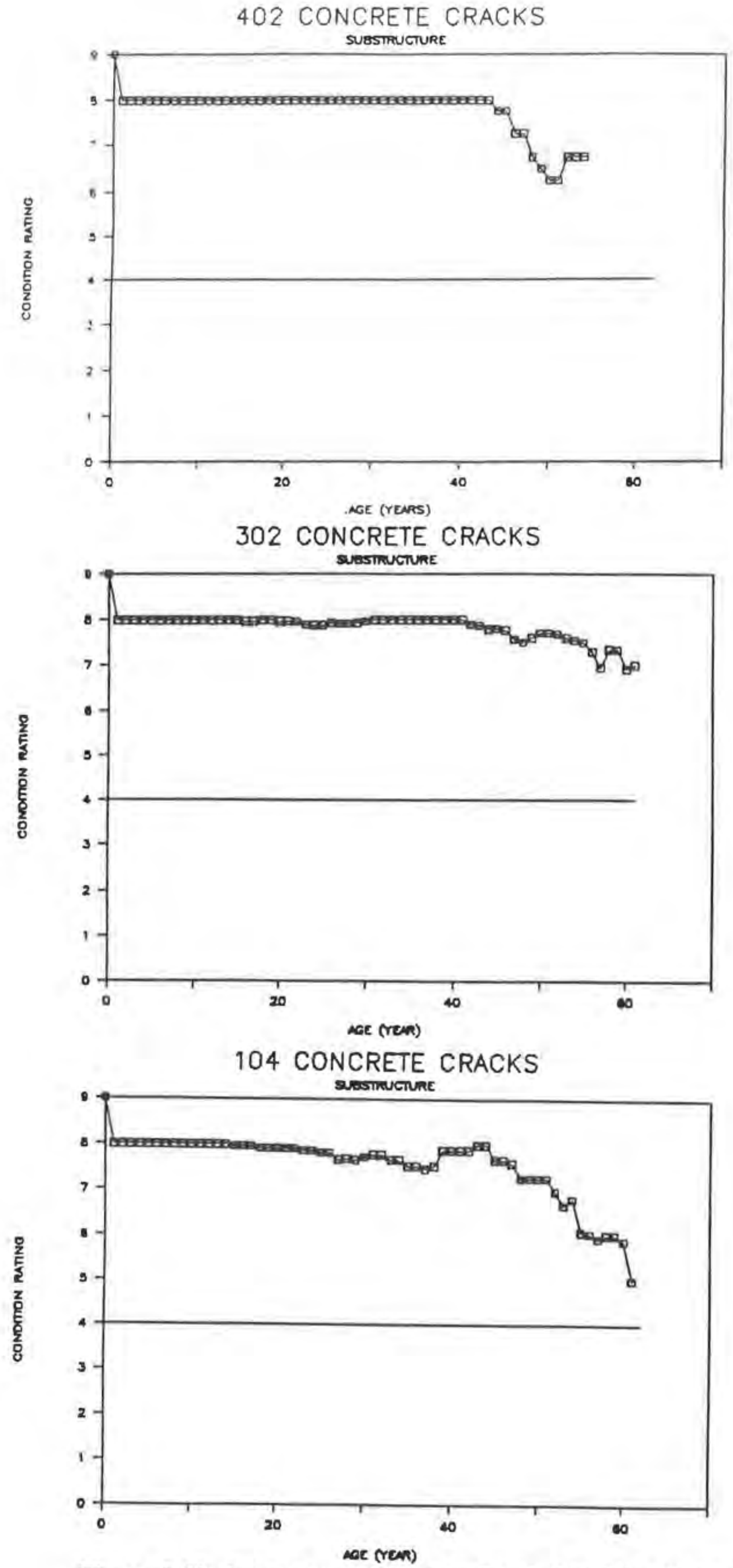


Figure 5.38 Substructure Concrete Crack Condition Rating vs Age Curves for 104, 302, 402 Bridges

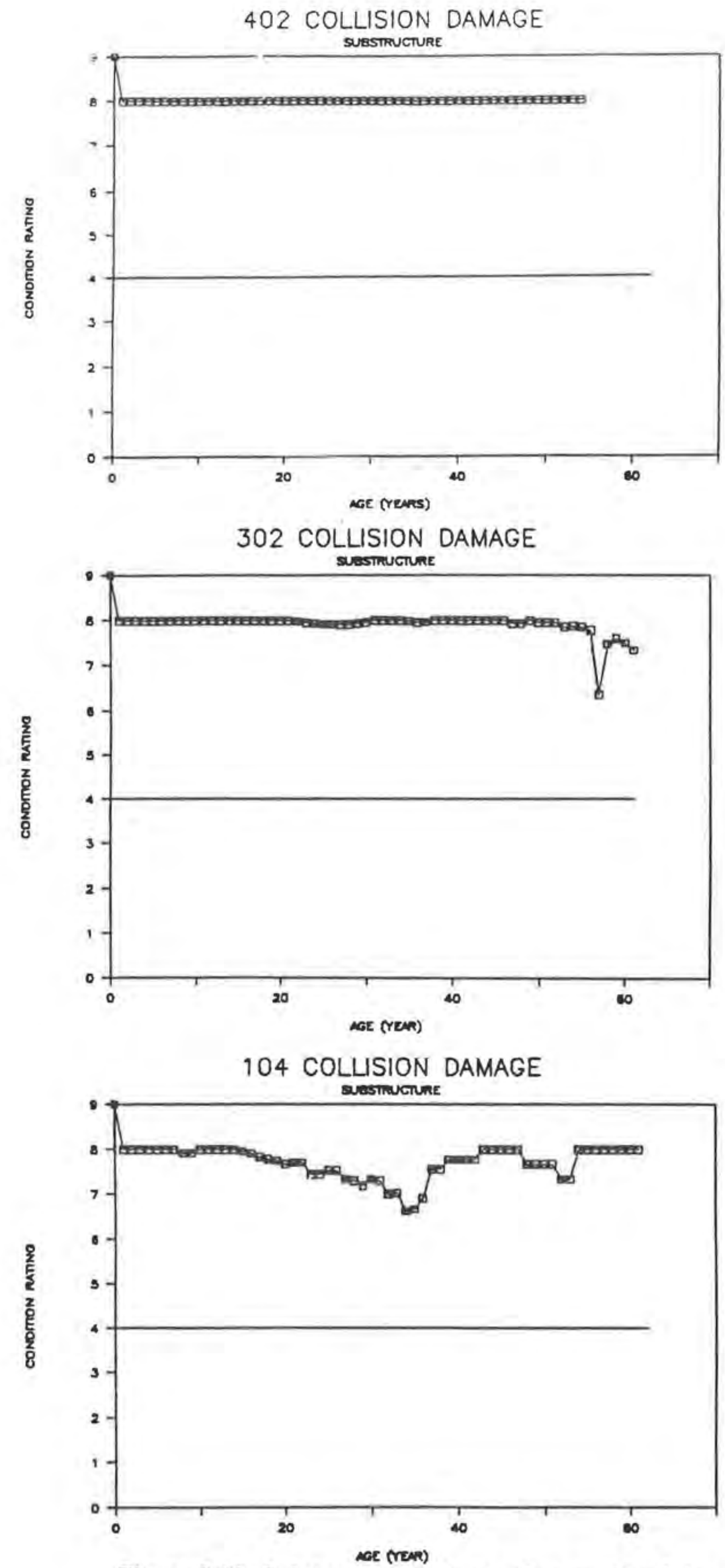


Figure 5.39 Substructure Collision Damage Condition Rating vs Age Curves for 104, 302, 402 Bridges

The age at first significant deterioration and the deterioration rate for each major component and subcomponent were determined from the curves of Figure 5.7-5.39 for each bridge type. The age at first significant deterioration was defined as the earliest age at which significant deterioration was found. This definition was expanded for concrete tee beam bridges (104) because eleven of these bridges were in service for only a relatively short period of time. Two of the tee beam bridges were replaced at age 25, which was the youngest age in the data set, and nine were in the age range of 26-35 years. The primary reason for replacing these bridges was because the deck geometries were not adequate. Five out of the eleven young bridge replacements were for this reason. One of the bridges was replaced because the underclearance was not adequate. Another, because the deck was not in good condition, and another because the substructure was not in good condition. Records for the remaining three bridges did not indicate reasons for the bridge replacements. The expanded definition of age at first significant deterioration (for concrete tee beam bridges only) is the age at which a definite continuing downward slope of deterioration occurs.

Component deterioration rates were determined by picking two points which were ten years apart and finding the condition rating that corresponded to the years picked. The difference between these two condition ratings divided by the difference in years gave the slope or component deterioration rate.

The age of first significant deterioration and the deterioration rate for each bridge component are shown in Tables 5.9, 5.10 and 5.11 for 302, 402, and 104 bridges, respectively. The condition ratings shown in Tables 5.9-5.11 are projected ratings rather than actual condition ratings. However, these were close to the actual condition ratings for those cases where checks could be made. These projected ratings were based on the assumption that the deterioration occurred linearly. Using the age at first significant deterioration and the deterioration rate, the authors were able to project the condition rating for 50, 60, 70, and 80 years using linear extrapolation. The columns in Tables 5.9-5.11 marked "Age at CR=4" give the projected age when subcomponent would be at a condition rating of 4 based on linear extrapolation. For convenience of comparison and for evaluating and grading the durability performances of the bridge subcomponents, the "Age at CR=4" column in Tables 5.9-5.11 were extracted and placed in Table 5.12.

Each subcomponent of the deck, superstructure, and substructure were evaluated based on its projected age at failure (CR = 4). These evaluations were made for three types of superstructure spans, i.e., concrete tee beams (104), steel stringer/multibeam or girder (302), or steel continuous stringer/multibeam of girder (402), with the aid of Table 5.12. Performance grades were given to each subcomponent based on the projected age at failure. If the component failed before an age of 45 years, it received an "F". If components failed between the ages of 45-59, 60-74, 75-89, they received a "D", "C", "B", respectively. If the component lasted longer than 90 years, it received an "A". The results of this evaluation/grading activity are presented in Table 5.13.

For most of the superstructure types, the subcomponent received similar ratings. There were a few areas, such as alignment of members, which had different ratings for different types of spans. The subcomponent that performed the worst (received a "D") were:

- Wearing surface
- Deck - Structural
- Curbs
- Expansion joints
- Deck's inspector's rating
- Paint

- Deflection under load
- Abutments - wings
- Abutments - backwall

The best performing subcomponents were:

- Drains
- Rivets or Bolts
- Welds - cracking
- Piers or bents - columns

Table 5.9
Component Performances of AU Subset 143
Steel Stringer/Multi-Beam or Girder (302)

COMPONENT	AGE AT FIRST SIGNIFICANT DETERIORATION	DETERIORATION RATE (CR/YR)	PROJECTED CONDITION RATING AT 50 YEARS	PROJECTED CONDITION RATING AT 60 YEARS	PROJECTED CONDITION RATING AT 70 YEARS	PROJECTED CONDITION RATING AT 80 YEARS	AGE AT CR = 4
==> DECK <==							
1. WEARING SURFACE	38	0.160	6.1	4.5	2.9	1.3	63
2. DECK – STRUCTURAL	36	0.165	5.7	4.0	2.4	0.7	60
3. CURBS	36	0.190	5.3	3.4	1.5	−0.4	57
4. RAILING	38	0.150	6.2	4.7	3.2	1.7	65
5. DRAINS	36	0.054	7.2	6.7	6.2	5.6	110
6. EXPANSION JOINTS / DEVICE	34	0.100	6.4	5.4	4.4	3.4	74
*** INSPECTOR'S RATING ***	33	0.150	5.5	4.0	2.5	1.0	60
==> SUPERSTRUCTURE <==							
1. BEARING DEVICES	35	0.090	6.7	5.8	4.9	4.0	79
2. STRINGERS, GIRDERS, & BEAMS	37	0.105	6.6	5.6	4.5	3.5	75
3. PAINT	37	0.130	6.3	5.0	3.7	2.4	68
4. RIVETS OR BOLTS	30	0.038	7.2	6.9	6.5	6.1	135
5. WELDS – CRACKING	41	0.119	6.9	5.7	4.5	3.4	75
6. RUST	35	0.110	6.4	5.3	4.2	3.1	71
7. COLLISION DAMAGE	41	0.034	7.7	7.4	7.0	6.7	159
8. DEFLECTION UNDER LOAD	34	0.120	6.1	4.9	3.7	2.5	67
9. ALIGNMENT OF MEMBERS	36	0.090	6.7	5.8	4.9	4.0	80
10. VIBRATION UNDER LOAD	37	0.130	6.3	5.0	3.7	2.4	68
*** INSPECTOR'S RATING ***	34	0.110	6.2	5.1	4.0	2.9	70
==> SUBSTRUCTURE <==							
1. ABUTMENTS – WINGS	36	0.165	5.7	4.0	2.4	0.7	60
– BACKWALL	39	0.120	6.7	5.5	4.3	3.1	72
– EROSION	40	0.075	7.3	6.5	5.8	5.0	93
– SETTLEMENT	40	0.070	7.3	6.6	5.9	5.2	97
2. PIERS OR BENTS – CAPS	36	0.105	6.5	5.5	4.4	3.4	74
– COLUMNS	35	0.070	7.0	6.3	5.6	4.9	92
– PILES	31	0.100	6.1	5.1	4.1	3.1	71
– SCOUR	39	0.065	7.3	6.6	6.0	5.3	101
– SETTLEMENT	31	0.050	7.1	6.6	6.1	5.6	111
3. PILE BENTS	34	0.085	6.6	5.8	4.9	4.1	81
4. CONCRETE CRACKS OR SPALLING	39	0.036	7.6	7.2	6.9	6.5	150
5. COLLISION DAMAGE	51	0.010	8.0	7.9	7.8	7.7	451
*** INSPECTOR'S RATING ***	35	0.230	4.6	2.2	−0.1	−2.4	52

Table 5.10
Component Performances of AU Subset 143
Steel Continuous Stringer/Multi-Beam or Girder (402)

COMPONENT	AGE AT FIRST SIGNIFICANT DETERIORATION	DETERIORATION RATE (CR/YR)	PROJECTED				AGE AT CR = 4
			CONDITION RATING AT 50 YEARS	CONDITION RATING AT 60 YEARS	CONDITION RATING AT 70 YEARS	CONDITION RATING AT 80 YEARS	
==> DECK <==							
1. WEARING SURFACE	40	0.280	5.4	2.8	0.2	−2.4	55
2. DECK – STRUCTURAL	40	0.250	5.5	3.0	0.5	−2.0	56
3. CURBS	40	0.140	6.6	5.2	3.8	2.4	69
4. RAILING	39	0.150	6.4	4.9	3.4	1.9	66
5. DRAINS	40	0.080	7.2	6.4	5.6	4.8	90
6. JOINT LEAKAGE	40	0.190	6.1	4.2	2.3	0.4	61
7. EXPANSION JOINTS / DEVICE	40	0.370	4.3	0.6	−3.1	−6.8	51
*** INSPECTOR'S RATING ***	40	0.285	5.2	2.3	−0.5	−3.4	54
==> SUPERSTRUCTURE <==							
1. BEARING DEVICES	40	0.150	6.5	5.0	3.5	2.0	67
2. STRINGERS, GIRDERS, & BEAMS	39	0.150	6.4	4.9	3.4	1.9	66
3. PAINT	40	0.210	5.9	3.8	1.7	−0.4	59
4. RIVETS OR BOLTS	40	0.110	6.9	5.8	4.7	3.6	76
5. RUST	40	0.070	7.3	6.6	5.9	5.2	97
6. CONCRETE CRACKING	40	0.170	6.3	4.6	2.9	1.2	64
7. COLLISION DAMAGE	40	0.180	6.2	4.4	2.6	0.8	62
8. DEFLECTION UNDER LOAD	40	0.235	5.7	3.3	1.0	−1.4	57
9. ALIGNMENT OF MEMBERS	40	0.035	7.7	7.3	7.0	6.6	154
10. VIBRATION UNDER LOAD	40	0.192	6.1	4.2	2.2	0.3	61
*** INSPECTOR'S RATING ***	40	0.200	6.0	4.0	2.0	0.0	60
==> SUBSTRUCTURE <==							
1. ABUTMENTS – WINGS	40	0.255	5.5	2.9	0.3	−2.2	56
– BACKWALL	33	0.223	4.2	2.0	−0.3	−2.5	51
– EROSION	34	0.182	5.1	3.3	1.4	−0.4	56
– SETTLEMENT	38	0.130	6.4	5.1	3.8	2.5	69
2. PIERS OR BENTS – CAPS	39	0.170	6.1	4.4	2.7	1.0	63
– COLUMNS	42	0.035	7.7	7.4	7.0	6.7	156
– PILES	37	0.165	5.9	4.2	2.6	0.9	61
– SCOUR	39	0.245	5.3	2.9	0.4	−2.0	55
– SETTLEMENT	40	0.310	4.9	1.8	−1.3	−4.4	53
3. PILE BENTS	39	0.260	4.9	2.1	−0.7	−3.5	53
4. CONCRETE CRACKS OR SPALLING	41	0.205	6.2	4.1	2.1	0.0	61
*** INSPECTOR'S RATING ***	39	0.240	5.4	3.0	0.6	−1.8	56

Table 5.11
Component Performances of AU Subset 143
Concrete Tee Beams (104)

COMPONENT	AGE AT FIRST SIGNIFICANT DETERIORATION	DETERIORATION RATE (CR/YR)	PROJECTED				AGE AT CR = 4
			CONDITION RATING AT 50 YEARS	CONDITION RATING AT 60 YEARS	CONDITION RATING AT 70 YEARS	CONDITION RATING AT 80 YEARS	
==> DECK <==							
1. DECK – STRUCTURAL	46	0.190	7.2	5.3	3.4	1.5	67
2. CURBS	42	0.165	6.7	5.0	3.4	1.7	66
3. RAILING	41	0.115	7.0	5.8	4.7	3.5	76
4. DRAINS	50	0.340	7.6	4.2	0.8	−2.6	61
5. EXPANSION JOINTS / DEVICE	41	0.180	6.4	4.6	2.8	1.0	63
*** INSPECTOR'S RATING ***	42	0.125	6.3	5.1	3.8	2.6	66
==> SUPERSTRUCTURE <==							
1. BEARING DEVICES	48	0.230	7.5	5.2	2.9	0.6	65
2. STRINGERS, GIRDERS, & BEAMS	44	0.165	6.5	4.9	3.2	1.6	65
3. PAINT	44	0.140	7.2	5.8	4.4	3.0	73
4. RIVETS OR BOLTS	46	0.120	7.5	6.3	5.1	3.9	79
5. WELDS – CRACKING	17	0.023	7.2	7.0	6.8	6.6	191
6. RUST	50	0.290	7.5	4.6	1.7	−1.2	62
7. CONCRETE CRACKING	50	0.155	7.4	5.9	4.3	2.8	72
8. COLLISION DAMAGE	47	0.200	7.4	5.4	3.4	1.4	67
9. DEFLECTION UNDER LOAD	47	0.260	7.2	4.6	2.0	−0.6	62
10. ALIGNMENT OF MEMBERS	45	0.163	7.2	5.6	3.9	2.3	70
11. VIBRATION UNDER LOAD	50	0.285	8.0	5.2	2.3	−0.5	64
*** INSPECTOR'S RATING ***	46	0.155	7.4	5.8	4.3	2.7	72
==> SUBSTRUCTURE <==							
1. ABUTMENTS – WINGS	42	0.135	6.9	5.6	4.2	2.9	72
– BACKWALL	42	0.180	6.6	4.8	3.0	1.2	64
– EROSION	44	0.145	7.1	5.7	4.2	2.8	72
– SETTLEMENT	46	0.120	7.5	6.3	5.1	3.9	79
2. PIERS OR BENTS – CAPS	42	0.120	7.0	5.8	4.6	3.4	75
– COLUMNS	45	0.100	7.5	6.5	5.5	4.5	85
– PILES	46	0.070	7.7	7.0	6.3	5.6	103
– SCOUR	45	0.130	7.4	6.1	4.8	3.4	76
– SETTLEMENT	46	0.150	7.4	5.9	4.4	2.9	73
3. CONCRETE CRACKS OR SPALLING	43	0.160	6.9	5.3	3.7	2.1	66
4. COLLISION DAMAGE	14	0.080	5.8	5.2	4.6	4.0	61
*** INSPECTOR'S RATING ***	42	0.160	6.7	5.1	3.5	1.9	67

Table 5.12
Projected Age at CR of 4 for AU
Subset 143 Bridge Components

Component	Projected Age at CR = 4		
	104 Bridges	302 Bridges	402 Bridges
Deck			
1. Wearing Surface	-	63	55
2. Deck - Structural	67	60	56
3. Curbs	66	57	69
4. Railing	76	65	66
5. Drains	61	110	90
6. Joint Leakage	-	-	61
7. Expansion Joints/Device	63	74	51
Inspector's Rating	68	60	54
Superstructure			
1. Bearing Devices	65	79	67
2. Stringers, Girders, & Beams	65	75	68
3. Paint	73	68	59
4. Rivets or Bolts	79	135	76
5. Welds - Cracking	191	75	-
6. Rust	62	71	97
7. Concrete Cracking	72	-	64
8. Collision Damage	67	159	62
9. Deflection Under Load	62	67	57
10. Alignment of Members	70	80	154
11. Vibration Under Load	64	68	61
Inspector's Rating	72	70	60
Substructure			
1. Abutments - Wings	72	60	56
- Backwall	64	72	51
- Erosion	72	93	56
- Settlement	79	97	69
2. Piers or Bents - Caps	75	74	63
- Columns	85	92	156
- Piles	103	71	61
- Scour	76	101	55
- Settlement	73	111	53
3. Pile Bents	-	81	53
4. Concrete Cracks or Spalling	68	150	61
5. Collision Damage	81	451	-
Inspector's Rating	67	52[1]	56

[1]Inspector's Rating here is inconsistent with subcomponent ratings.

Table 5.13
Longevity Performance Grade for AU Subset
143 Bridge Components

COMPONENT	LONGEVITY PERFORMANCE GRADE 104 BRIDGES	302 BRIDGES	402 BRIDGES
==> DECK <==			
1. WEARING SURFACE	–	C	D
2. DECK – STRUCTURAL	C	C	D
3. CURBS	C	D	C
4. RAILING	B	C	C
5. DRAINS	C	A	A
6. JOINT LEAKAGE	–	–	C
7. EXPANSION JOINTS / DEVICE	C	C	D
*** INSPECTOR'S RATING ***	C	C	D
==> SUPERSTRUCTURE <==			
1. BEARING DEVICES	C	B	C
2. STRINGERS, GIRDERS, & BEAMS	C	B	C
3. PAINT	C	C	D
4. RIVETS OR BOLTS	B	A	B
5. WELDS – CRACKING	A	B	–
6. RUST	C	C	A
7. CONCRETE CRACKING	C	–	C
8. COLLISION DAMAGE	C	A	C
9. DEFLECTION UNDER LOAD	C	C	D
10. ALIGNMENT OF MEMBERS	C	B	A
11. VIBRATION UNDER LOAD	C	C	C
*** INSPECTOR'S RATING ***	C	C	C
==> SUBSTRUCTURE <==			
1. ABUTMENTS – WINGS	C	C	D
– BACKWALL	C	C	D
– EROSION	C	A	D
– SETTLEMENT	B	A	C
2. PIERS OR BENTS – CAPS	B	C	C
– COLUMNS	B	A	A
– PILES	A	C	C
– SCOUR	B	A	D
– SETTLEMENT	C	A	D
3. PILE BENTS	–	B	D
4. CONCRETE CRACKS OR SPALLING	C	A	C
5. COLLISION DAMAGE	B	A	–
*** INSPECTOR'S RATING ***	C	D	D

GRADE BASED ON PROJECTED AGE AT CR = 4

AGE AT CR = 4	PERFORMANCE GRADE
BELOW 45	F UNSATISFACTORY
45 – 59	D FAIR
60 – 74	C SATISFACTORY/GOOD
75 – 89	B VERY GOOD
90 AND ABOVE	A EXCELLENT

5.6 Comparison of Concrete and Steel Bridge Durability Performances

The graphs for the various components of the bridges that were generated from the AU Subset 143 in Figures 5.7-5.39 were used to compare the longevity performances of components of simple span steel girder bridges (302) verses simple span concrete tee beam bridges (104). Bridge deterioration data in Tables 5.9 and 5.11 were placed side-by-side in Table 5.14 for convenience of comparison. It should be noted that more data was available for the steel bridges (80 steel versus 24 concrete bridges).

Table 5.14
Comparison of Longevity Performances of Concrete Simple Span Tee Beam (104) Steel Simple Span Girder (302) Bridges for AU Subset 143 Bridges

COMPONENT	AGE AT FIRST SIGNIFICANT DETERIORATION		DETERIORATION RATE (CR/YR)		AGE AT CR = 4	
	104	302	104	302	104	302
==> DECK <==						
1. WEARING SURFACE	–	38	–	0.160	–	63
2. DECK – STRUCTURAL	46	36	0.190	0.165	67	60
3. CURBS	42	36	0.165	0.190	68	57
4. RAILING	41	38	0.115	0.150	76	65
5. DRAINS	50	36	0.340	0.054	61	110
6. EXPANSION JOINTS / DEVICE	41	34	0.180	0.100	63	74
*** INSPECTOR'S RATING ***	42	33	0.125	0.150	68	60
==> SUPERSTRUCTURE <==						
1. BEARING DEVICES	48	35	0.230	0.090	65	79
2. STRINGERS, GIRDERS, & BEAMS	44	37	0.165	0.105	65	75
3. PAINT	44	37	0.140	0.130	73	68
4. RIVETS OR BOLTS	46	30	0.120	0.038	79	135
5. WELDS – CRACKING	17	41	0.023	0.119	191	75
6. RUST	50	35	0.290	0.110	62	71
7. CONCRETE CRACKING	50	–	0.155	–	72	–
8. COLLISION DAMAGE	47	41	0.200	0.034	67	159
9. DEFLECTION UNDER LOAD	47	34	0.280	0.120	62	67
10. ALIGNMENT OF MEMBERS	45	38	0.163	0.090	70	80
11. VIBRATION UNDER LOAD	50	37	0.285	0.130	64	68
*** INSPECTOR'S RATING ***	46	34	0.155	0.110	72	70
==> SUBSTRUCTURE <==						
1. ABUTMENTS – WINGS	42	38	0.135	0.165	72	60
– BACKWALL	42	39	0.180	0.120	64	72
– EROSION	44	40	0.145	0.075	72	93
– SETTLEMENT	46	40	0.120	0.070	79	97
2. PIERS OR BENTS – CAPS	42	38	0.120	0.105	75	74
– COLUMNS	45	35	0.100	0.070	85	92
– PILES	46	31	0.070	0.100	103	71
– SCOUR	45	39	0.130	0.065	76	101
– SETTLEMENT	46	31	0.150	0.050	73	111
3. PILE BENTS	–	34	–	0.085	–	81
3. CONCRETE CRACKS OR SPALLING	43	39	0.160	0.036	68	150
4. COLLISION DAMAGE	14	51	0.080	0.010	81	451
*** INSPECTOR'S RATING ***	42	35	0.180	0.230	67	52

An initial examination of the tables and figures referenced above indicate that the steel bridges had a greater longevity. Some general observations of these data are as follows:

- The steel bridges tended to exhibit signs of deterioration at an earlier age.
- Typically, the concrete bridges had a higher deterioration rate.
- Most subcomponents of the concrete bridges reached failure (CR=4) at an earlier age.

Comparisons of the three major components and subcomponents are presented below. As will be seen, strong conclusions can not be made because of inconsistencies in the data. However, some valuable insights and information can be obtained by examining the data.

5.6.1 Decks

Most deck subcomponents for steel bridges exhibited earlier signs of deterioration than the concrete bridges, but lower deterioration rates. Drains and expansion joints for steel bridges exhibited much lower deterioration rates than their concrete counterparts. The deterioration rate for drains for the concrete bridges was 0.340, and 0.054 for the steel bridges. This corresponded to a fifty years difference in life expectancy in favor of the steel bridges. The performance of drains should be almost independent of the bridge superstructure material. Hence, any differences in performance of this item should be attributed to drain design, construction, or maintenance differences.

The lower deterioration rate for expansion joints for steel bridges resulted in an 11 year longer life expectancy. Steel bridge spans are typically longer than those of concrete, and hence, their expansion joint gaps are made wider. The performance of expansion joints are primarily evaluated on the basis of this gap or spacing. Consequently, the fact that the steel bridge expansion joints were evaluated to perform better than those for concrete bridges was not surprising.

The somewhat lower deterioration rates of the concrete bridges for decks, curbs, and railings, which are basically the same for both bridge superstructure materials, are probably attributable to reduced stiffness of the steel superstructure bridge. This results in greater bridge deflections, and in turn, greater concrete deck, curb, etc., cracking and deteriorations.

The first significant deterioration of the overall inspector's rating for decks for concrete bridges was forty-two years, and for steel bridges was thirty-three years. These, coupled with deterioration rates of 0.125 for concrete and 0.150 for steel bridges, resulted in a difference in life expectancy in favor of the concrete bridges of eight years.

5.6.2 Superstructures

Steel span bearing devices performed significantly better than their concrete bridge counterparts'. The first significant deterioration of bearing devices for steel bridges was at thirty-five years, whereas for concrete bridges, it was forty-eight years. However, the concrete deterioration rate of 0.230 was much larger than the 0.090 of the steel bridges. The net result was a fourteen year difference in the life expectancy in favor of the steel spans.

Steel spans also performed better for the stringers, girders, and beams. The age at first significant deterioration of the stringers, girders, and beams was forty-four years for concrete span bridges and thirty-seven years for steel bridges. The deterioration rates of the stringers, girders, and beams were 0.165 and 0.105 for the concrete and steel span bridges, respectively. These rates corresponded to a ten year difference in the life expectancy in favor of steel bridges. The difference in deterioration rate is probably related to the deterioration caused by the cracks formed in the concrete due to the applied loads. These cracks allow water to enter and corrode the rebar, and in turn, result in a higher deterioration rate for the concrete members.

Paint, rust, rivets or bolts, and weld-cracking are bridge superstructure subcomponents which are basically unique to steel bridges. Appropriate and meaningful comparisons of these subcomponents between steel and concrete bridges are not possible. Likewise concrete superstructure cracking is unique to concrete bridges. However, the absolute performances of these items which are unique to a particular bridge type should be examined as they directly reflect on the durability/ longevity character for that bridge type.

Paint is a very important item to the superstructure longevity performance of steel bridges, since deterioration of the paint will expose the steel leaving it susceptible to rust and corrosion. The age of the first significant deterioration of the paint for the steel bridges was thirty-seven years, and the deterioration rate was 0.130. These combine to yield a life expectancy of 68 years. This is an area where proper maintenance should be able to greatly extend the life expectancy.

Bridge performance evaluations for collision damage as depicted in the AU Subset 143 are somewhat suspect. Concrete and steel simple span bridges exhibited projected failure ages due to collisions of 67 and 159 years respectively, whereas continuous span steel bridges had a projected failure age from collisions of 62 years. The first set of numbers would imply that steel bridge collision performances were far superior to those of concrete. However, the second set of numbers indicated that simple span steel bridge collision performances were far superior to continuous span steel bridges. In terms of catastrophic damages, this latter indication is contrary to what we would expect based on theory. It is most probable that the low incidences of collision and the nature of the collision make the dataset too small to assess relative performances. For example, from theory, we would expect that a collision involving a fire would probably be more detrimental to a steel bridge. On the other hand, steel is a tougher and more ductile material, and we would expect it to withstand collisions and impact loadings better than concrete.

Since steel bridges typically have longer spans and smaller flexural rigidities (EI), one would expect the steel bridges to exhibit greater deflections. Creep will cause concrete bridges to deflect with constant loading over a long period of time. However, this additional creep deflection is rather small. Again, steel bridges exhibited earlier significant deterioration, but concrete bridges exhibited the higher rate of deterioration. The net results were that bridges of both materials failed at approximately the same age in this category.

Steel span bridges performed better for alignment of members than the concrete spans. Concrete bridges usually have more joints, and joints are where members come out of alignment. Hence, concrete bridges are more susceptible to falling out of alignment.

The vibration under load component's deterioration is directly related to the deflection under load component. For traffic loadings, the more flexible the span, the greater the vibration amplitudes. The deterioration rates of deflection under load component were very close to the deterioration rates for vibration under load, as were the ages at first significant deterioration. The overall inspector's rating for superstructures exhibited the same pattern of earlier age of first significant deterioration for steel bridges and higher rate of deterioration for concrete bridges. The net result was approximately the same age at failure for both material types.

5.6.3 Substructures

Theory would indicate that substructure performances should not be strongly dependent on the type of superstructure. However, superstructure parameters having the greatest impact on substructure performance would probably be the weight and vibrational characteristics. Heavier concrete superstructure would be more demanding on the substructure. Offsetting this to some degree, however, would be the greater vibration induced superstructure support displacements and load variations transmitted to the substructures of the more flexible steel bridges.

The CR vs. Age plots indicate that the overall performances of concrete bridge substructures were somewhat better than for steel bridges. The age of first significant deterioration of the inspector's rating of the substructure for the concrete spans was forty-two years, and thirty-five years for the steel spans. The deterioration rate of substructure's inspector's rating for concrete spans was 0.160, and the rate for steel spans was 0.230. This corresponded to concrete spans reaching failure at an age of sixty-seven years compared with the steel spans reaching failure at an age of fifty-two years.

The wings of the abutments performed better for concrete spans than steel spans, exhibiting a life expectancy difference of twelve years in favor of concrete spans. However, the backwall of the abutments performed better for steel spans than for concrete spans, exhibiting a life expectancy difference of eight years in favor of steel spans. One would anticipate that the whole abutment would perform about the same for each bridge/material type. This was not the case, and the difference is probably due to data inprecision rather than the bridge superstructure material.

Possible reasons for this inconsistency were because the wings of the abutments would tend to fail due to local geotechnical and drainage area conditions rather than the type of material in the bridge's superstructure. However, the backwall of the abutment would probably be more susceptible to the bridge superstructure material. Thermal expansions of the bridge, in association with joints filling with dirt/debris (with concrete bridges tending to have the greater number of joints), could be the source of abutment backwall deteriorations causing a lower lifespan of the concrete span.

Erosion performance is not a function of the type of material used in the bridge's superstructure. However, there is a correlation between erosion and settlement, i.e., the greater the erosion, the more the abutment will be able to move and settle. This can be seen by examining the graphs for the erosion and settlement. The age at first significant deterioration of erosion for concrete spans was forty-four years; and for the settlement, it was forty-six years. The ages at first significant deterioration were forty years for both the erosion and settlement of abutments for steel spans. The deterioration rate of erosion for concrete spans was 0.145, and was 0.120 for the settlement. The deterioration rates of erosion and settlement for steel spans were 0.075 and 0.070, respectively. These rates are quite similar. When the erosion component of the concrete bridges reached failure at an age of seventy-two years, the settlement component reached failure soon after at an age of seventy-nine years. Settlement of the abutment and piers probably caused other bridge components to deteriorate more rapidly due to enhanced cracking and traffic impact loadings.

The caps of the piers or bents performed the same for either type of material, with failure occurring at about seventy-five years for both materials.

The columns of piers performed somewhat better for steel spans than for concrete spans. Concrete spans reached failure at age eighty-five years, and steel spans reached failure at age ninety-two years. This small difference in projected failure age is probably not significant. However, the lighter steel bridges should enhance pier/bent column performance.

Concrete spans performed better for piles of the piers than steel spans, having a thirty year difference in the life expectancy in favor of the concrete span. First thoughts would be that pile performances should be about the same as pier/bent columns. However, the greater flexibility and vibration levels for steel bridges probably result in additional pile displacements for steel bridges.

The scouring component should not perform differently with the different type of material, but as with the erosion of the abutment, it does enhance settlement. When the scouring reached failure, the settlement reached failure a short time later. Concrete bridge age at first significant deterioration of scouring was forty-five years, and forty-six years for the settlement of piers or bents. Steel bridges' age at first significant deterioration was thirty-nine years for the scouring, and thirty-one years for the settlement of the piers. The deterioration rate of the scour component for concrete spans was 0.130, and 0.150 for pier settlement. The deterioration rate of the scour component for steel spans was 0.065, and 0.050 for pier settlement. This corresponded to the concrete spans failing at an age of seventy-six years for the scour component, and seventy-three years for pier settlement. Also, this corresponded to the steel spans failing at an age of 101 years for the scour component, and 111 years for the pier settlement component.

Steel bridges performed better for substructure concrete cracking than concrete bridges. The heavier concrete bridges had a deterioration rate in excess of 4 times that of steel bridges, and a life of 68 years versus 150 years for steel bridges. Since the concrete bridges exhibited greater support settlements, then it is understandable that they would also exhibit more cracking.

Substructure collision damage is not strongly related to the superstructure material. The vast difference in performance for this item was probably related to the low number of such damages, and the laws of chance on what happened to be the bridge superstructure material when an incidence of a collision with a bridge substructure element occurred. It should be noted, however, that, shorter span concrete bridges require more bent/pier supports thus making them more susceptible to substructure collision damage.

5.7 Comparison of Continuous and Simple Span Durability Performances

The AU Subset 143 data contained bridge performance data on seventy continuous steel stringer/multi-beam or girder bridges (402 code number), and twenty-two simple span steel stringer/multi-beam or girder bridges (302 code number). Comparison of the performances of these two data sets affords the opportunity to assess the relative performance of continuous verses simple span construction. This comparison is presented below and is based on bridge performances in each category as reflected in the Condition Ratings (CR) verses Age curves of Figures 5.7 - 5.39. For convenience of comparison the bridge deterioration data of Tables 5.9 and 5.10 are placed side-by-side in Table 5.15.

Some general comparisons between simple and continuous span performances that can be made by examining the curves and tables referred to above are as follows:

- The vast majority of simple span components showed signs of deterioration before the continuous spans.
- The components of continuous spans generally had a steeper deterioration slope than did the simple spans.
- The greater deterioration slope caused the continuous span components to reach condition ratings of 4 first, i.e., continuous spans usually failed before the simple spans.

Based on these comparisons, one would conclude that simple spans perform better than continuous spans for the data sets studied in this report. Comparisons of the performances of the three major bridge components and their various subcomponents are presented below.

Table 5.15
Comparison of Longevity Performances of Simple Span (302) and Continuous Steel Bridge (402) for AU Subset 143 Bridges.

COMPONENT	AGE AT FIRST SIGNIFICANT DETERIORATION		DETERIORATION RATE (CR/YR)		AGE AT CR = 4	
	302	402	302	402	302	402
==> DECK <==						
1. WEARING SURFACE	38	40	0.160	0.260	63	55
2. DECK – STRUCTURAL	36	40	0.165	0.250	60	56
3. CURBS	36	40	0.190	0.140	57	69
4. RAILING	38	39	0.150	0.150	65	66
5. DRAINS	36	40	0.054	0.080	110	90
6. JOINT LEAKAGE	–	40	–	0.190	–	61
7. EXPANSION JOINTS/DEVICES	34	40	0.100	0.370	74	51
*** INSPECTOR'S RATING ***	33	40	0.150	0.285	60	54
==> SUPERSTRUCTURE <==						
1. BEARING DEVICES	35	40	0.090	0.150	79	67
2. STRINGERS, GIRDERS, & BEAMS	37	39	0.105	0.150	75	66
3. PAINT	37	40	0.130	0.210	68	59
4. RIVETS OR BOLTS	30	40	0.038	0.110	135	76
5. WELDS – CRACKING	41	–	0.119	–	75	–
6. RUST	35	40	0.110	0.070	71	97
7. CONCRETE CRACKING	–	40	–	0.170	–	64
8. COLLISION DAMAGE	41	40	0.034	0.180	159	62
9. DEFLECTION UNDER LOAD	34	40	0.120	0.235	67	57
10. ALIGNMENT OF MEMBERS	36	40	0.090	0.035	80	154
11. VIBRATION UNDER LOAD	37	40	0.130	0.192	68	61
*** INSPECTOR'S RATING ***	34	40	0.110	0.200	70	60
==> SUBSTRUCTURE <==						
1. ABUTMENTS – WINGS	36	40	0.165	0.255	60	58
– BACKWALL	39	33	0.120	0.223	72	51
– EROSION	40	34	0.075	0.162	93	56
– SETTLEMENT	40	38	0.070	0.130	97	69
2. PIERS OR BENTS – CAPS	36	39	0.105	0.170	74	63
– COLUMNS	35	42	0.070	0.035	92	156
– PILES	31	37	0.100	0.165	71	61
– SCOUR	39	39	0.065	0.245	101	55
– SETTLEMENT	31	40	0.050	0.310	111	53
3. PILE BENTS	34	39	0.085	0.280	81	53
4. CONCRETE CRACKS OR SPALLING	39	41	0.036	0.205	150	61
5. COLLISION DAMAGE	51	–	0.010	–	451	–
*** INSPECTOR'S RATING ***	35	39	0.230	0.240	52	56

5.7.1 Decks

The deck is a typical example of the general comparison results stated above. All of the deck components of the simple span bridges began to deteriorate before the corresponding components of the continuous bridges. But, most of the continuous spans, had a higher deterioration slope which dominated and caused each of the continuous components to reach a condition rating of 4 before the simple span construction. Note that the curb component is the one exception to this.

The deck wearing surface is given a condition rating based on visual inspection and smoothness of the ride across the deck. If any rough surfaces or potholes exist to cause bumpiness, then the condition rating is lowered. Bumpiness or rough surfaces can be caused by relative movements of members of the bridge, expansion and contraction of the deck itself, and by surface material deterioration due to traffic and environmental loads. The wearing surfaces reached failure at ages of 63 years and 55 years, respectively, for the simple and continuous spans. This somewhat poorer performance of the continuous spans' wearing surface was probably related to the relative higher level of deck cracking and movement caused by traffic and thermal loadings for continuous span decks.

The structural part of the deck is also effected by the thermal expansion and contraction problems. Structural deck durability/longevity performances for simple span versus continuous span were approximately the same as for the wearing surfaces.

Curbs are the component of the deck that do not follow the general comparisons stated earlier. The reason for the continuous spans superior performance for the curb component was probably related to bridge alignment. That is, members of simple span bridges tend to get out of alignment easier than do continuous bridges simply because there are more pieces to keep aligned for the simple span construction. If the curbs get out of alignment, they are more susceptible to damage from debris build-up, improper drainage, and traffic. Also, they are given lower condition ratings when they are out of alignment.

There does not appear to be a difference between simple and continuous spans performance of the railing component. Their deterioration rates were the same, and the ages that they began to show deterioration were the same.

The drains for both types of the spans have a long life expectancy. Simple spans had a life of 110 years, and continuous spans had a life of 90 years. These are the life expectancies that ALDOT would like to achieve for all bridge components. Drain condition ratings are assessed based on how adequately water can flow through the drains. If the drains are out of alignment and/or are clogging up, then they restrain the passage of water. The deck drains' data for both simple and continuous spans indicated that the drains were working well and as designed. It appears that the drain openings were large enough for the water to be removed from the deck. The drains were one of the best performing components on bridges for both simple and continuous spans.

The expansion joints follow the general comparisons mentioned earlier. The deterioration rate for expansion joints for simple spans was 0.100, and for continuous spans, it was 0.370. The primary reasons for the superior performance of simple span construction in this area were probably as follows. Continuous spans are typically longer and naturally would be effected by the problems of expansion and contraction more than shorter simple spans. Also, since simple span bridges have more joints/expansion joints, then there would tend to be more total open space between spans for the entire length of the bridge. Expansion joints are given a condition rating based on alignment and openings between the joints. If there is not adequate open space or the joints do not line up, then a low condition rating is assigned. Simple span construction results in more total openings along the bridge than do continuous spans, and thus one would expect that expansion joint ratings would be higher for simple span bridges.

5.7.2 Superstructures

By examining the inspector's rating for the superstructures, one can see that simple spans performed somewhat better than continuous spans. The simple spans have a deterioration rate of 0.110, and the continuous spans have a deterioration rate of 0.200 for the inspector's rating of the superstructure. This corresponded to a ten year difference in the bridge's life expectancy between simple and continuous span construction for the overall superstructure.

Simple span bearing device components performed better than their continuous span counterparts, exhibiting a twelve year greater life expectancy. This is as one would expect unless special provisions are made in the devices design for greater mean loads, load flucations, and number of loading cycles for continuous span construction. Continuous spans are normally longer, and hence dead and live loads going to the bearing devices are larger. Loadings anywhere on continuous bridges affect the bearing device loads and movements. Also, thermal loadings on continuous bridges cause significantly greater vertical load flucations and longitudinal movements at bearing devices than they do for simple span construction. Since mean stresses, stress flucations, stress cycles and device movements are all typically larger for continuous bridge bearing devices, it is expected that their fatigue/wear/deterioration rates will be higher.

Bridge stringers, girders, and beams performed better for simple spans than for continuous spans for about the same reasons as for the bearing devices.

Components like paint, weld-cracking, collision damage, and rust will not be discussed here because there was not sufficient information to make comparisons. Also, the deterioration of parameters such as these (except for weld-cracking) are not strongly related to the type of bridge construction, i.e., simple or continuous span.

The rivets or bolts component follows the general comparisons that were stated previously. The deterioration rate for simple spans was 0.038 for the rivets or bolts, and for continuous spans, it was 0.110. These rates correspond to a sixty-one year life expectancy difference in favor of simple span construction. The reasons for the continuous spans deteriorating more rapidly are basically the same as those stated above for the more rapid deterioration for the continuous bridge bearing devices.

The simple span bridge performances in the area of deflection under load were superior to those of continuous span bridges, resulting in a ten year increase in life expectancy. The primary reason for the lower deflection ratings for continuous bridges was probably the longer span lengths used with these bridges. Bridge moments and stresses typically vary as the square of span length and stringers/girders are sized accordingly; however, deflections vary as the length to the fourth power and hence are normally much larger for longer spans.

A seventy-four year life expectancy difference in favor of the continuous span construction was found for alignment of members. By definition of continuous span construction, this is as one would expect.

Bridge vibration under load assessments followed the general comparisons, with a difference in life expectancy of about five years in favor of simple spans. It appears that the shorter simple span bridges, which are stiffer and have higher natural vibration frequencies than the longer continuous span bridges, performed and/or were rated slightly superior.

5.7.3 Substructures

Components of the substructure follow the general comparisons cited earlier except for the columns of the piers or bents. By examining the graphs for each component of the substructure, simple spans performed better than continuous spans. However, if one examines the graph of the inspector's rating, then the continuous span performs better. The reasons for this incompatibility are not known.

Comparing the abutments of the simple spans verses the continuous spans, the simple spans performed better. Again, greater loads with greater flucations and a greater number of cycles of flucation in conjunction with greater longitudinal loadings and movements, all contribute to a real potential for more rapid degradation of support abutments in continuous bridges.

Further, continuous spans had greater abutment settlements than simple spans, and the life expectancy was 28 years greater for simple spans. Generally, the primary reasons for this are the same as those cited above for the abutment/wings.

Pier or bent components mostly follow the general comparisons cited earlier except for the columns of the bents or piers. The continuous spans deterioration rate was 0.035 for the columns, and for simple spans, the rate was 0.070. This lower rate for continuous spans reflected a superior performance of continuous spans in this area. A possible reason for the superior performance of bents or pier columns in continuous bridges was the reduced impact loading on these elements due to the reduced number of bridge transverse joints in the deck. Repetitive impact loadings from bridge truck traffic were a source of fatigue stress on several bridge components, including the bent/piers columns.

The deterioration rate for the simple spans for pier/bent caps was 0.105, while being 0.170 for continuous spans. Pier/bent pile deterioration rate for simple spans was 0.100, while being 0.165 for continuous spans. These deterioration rates implied that simple spans out-perform continuous spans in this area. Since piers or bents should act somewhat similar to the bridge abutments, the same reasons cited for superior abutment performances are equally applicable to piers or bents.

Scour and erosion performances will not be discussed here since the performances of these factors are not primary functions of the type of bridge span construction, i.e., simple verses continuous.

The data indicated that piers and bents settle more rapidly with continuous spans just as they did for abutments. The reasons for the more rapid settlement for piers or bents for continuous bridges were the same as for settlement of abutments.

As expected, the pile bents performed similar to the piers or bents, with a twenty-eight year difference in life expectancy between simple and continuous spans, in favor of simple spans. The reasons stated for simple spans performing better for the piers or bents also apply to the pile bents.

Concrete cracks followed the general comparisons, also. The deterioration rate for simple spans was 0.036 for concrete crack component; and for continuous spans, it was 0.205. These resulted in a 89 year difference in life expectancy in favor of simple spans.

5.8 Comparison of Short, Intermediate, Long Span Bridge Durability Performances

The AU Subset data was separated based on bridge span lengths to compare the durability performances of the major bridge components based on span length. For purposes of making this comparison, three span lengths were chosen as follows:

- Short Span $L < 30$ (74 Bridges)
- Intermediate Span $30 \leq L \leq 80$ (42 Bridges)
- Long Span $L > 80$ (8 Bridges)

The number of bridges in each category is shown in parenthesis, with the predominate span length being short, and with a small population of long span bridges in the dataset.

Figure 5.40, 5.41, and 5.42 show the data plotted as CR vs Age curves for each major bridge component for long spans, intermediate spans and short spans, respectively. These same data were also plotted in an alternate form for convenience of comparisons as shown in Figures 5.43 - 5.45. In these latter figures, CR vs Age curves are shown for the three different span lengths for bridge decks (Figure 5.43), superstructures (Figure 5.44), and substructures (Figure 5.45). Observations and comments on the component durability performances based on Figure 5.40 - 5.45 are given below.

5.8.1 Decks

The dataset indicates early initiation of deck damage for long span bridges, and an overall improved durability performance with reduction in span length, i.e., the shorter bridge decks performed best and were followed by intermediate span bridges. Long span bridges exhibited early deteriorations and slightly larger deterioration-rates starting at an age of approximately 40 years.

5.8.2 Superstructures

Figures 5.40 - 5.42 indicate that bridge superstructures performed the best of the 3 major components for short spans and the worst for long spans. This is as one would expect. Both the deck and superstructure durability performances were better for short spans, and would indicate usage of short spans wherever feasible to enhance durability performance.

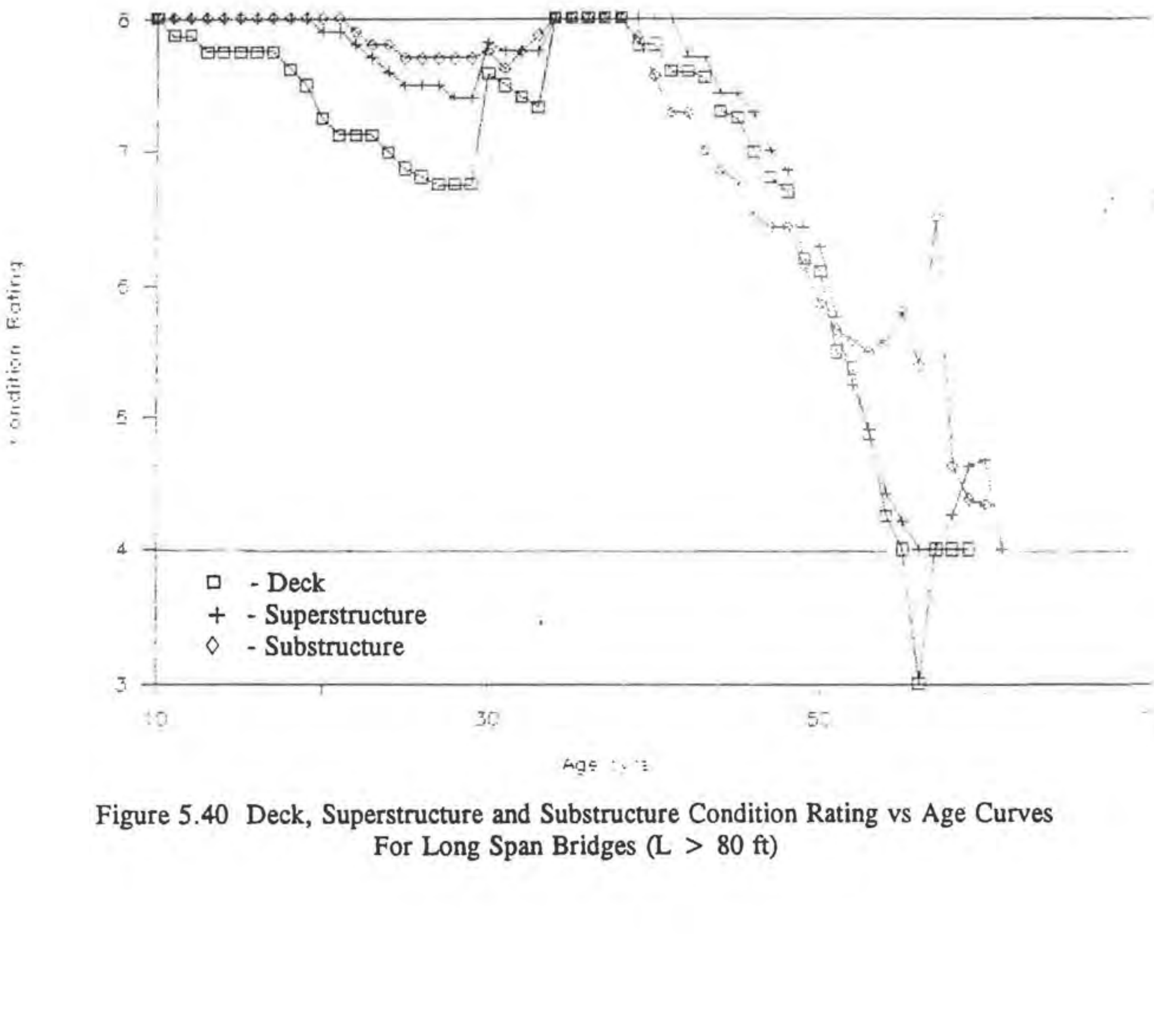


Figure 5.40 Deck, Superstructure and Substructure Condition Rating vs Age Curves For Long Span Bridges (L > 80 ft)

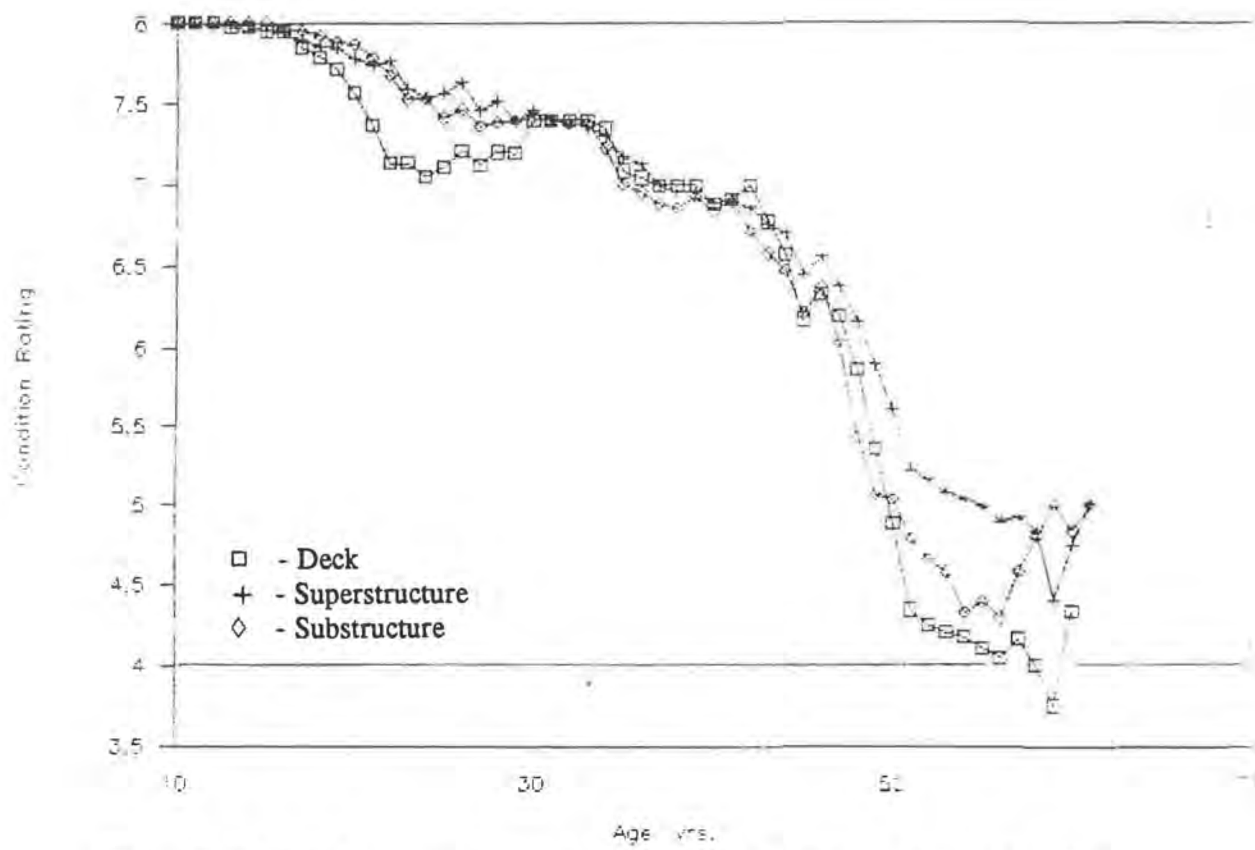


Figure 5.41 Deck, Superstructure and Substructure Condition Rating vs Age Curves For Intermediate Span Bridges (30 ≤ L ≤ 80 ft)

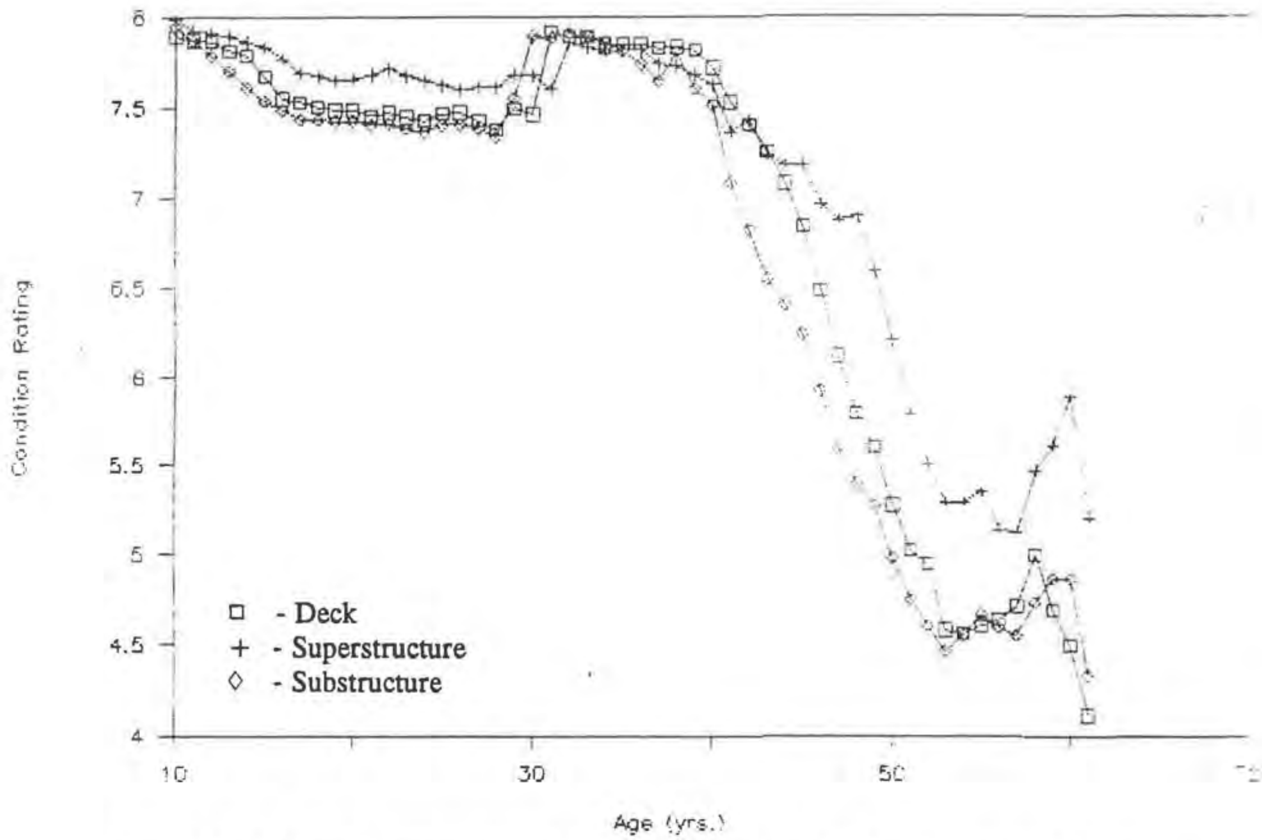


Figure 5.42 Deck, Superstructure and Substructure Condition Rating vs Age Curves For Short Span Bridges (L < 30 ft)

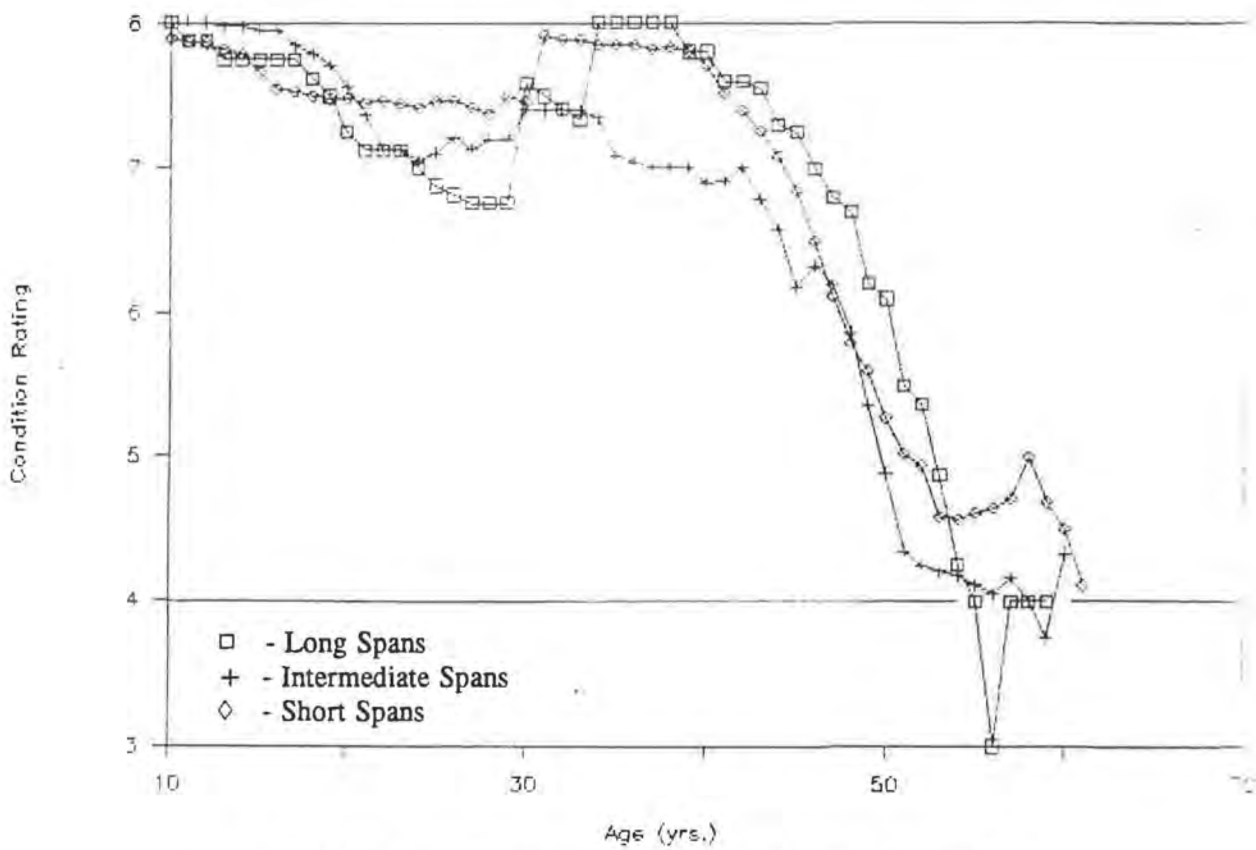


Figure 5.43 Deck Condition Rating vs Age Curves for Short, Intermediate and Long Span Bridges

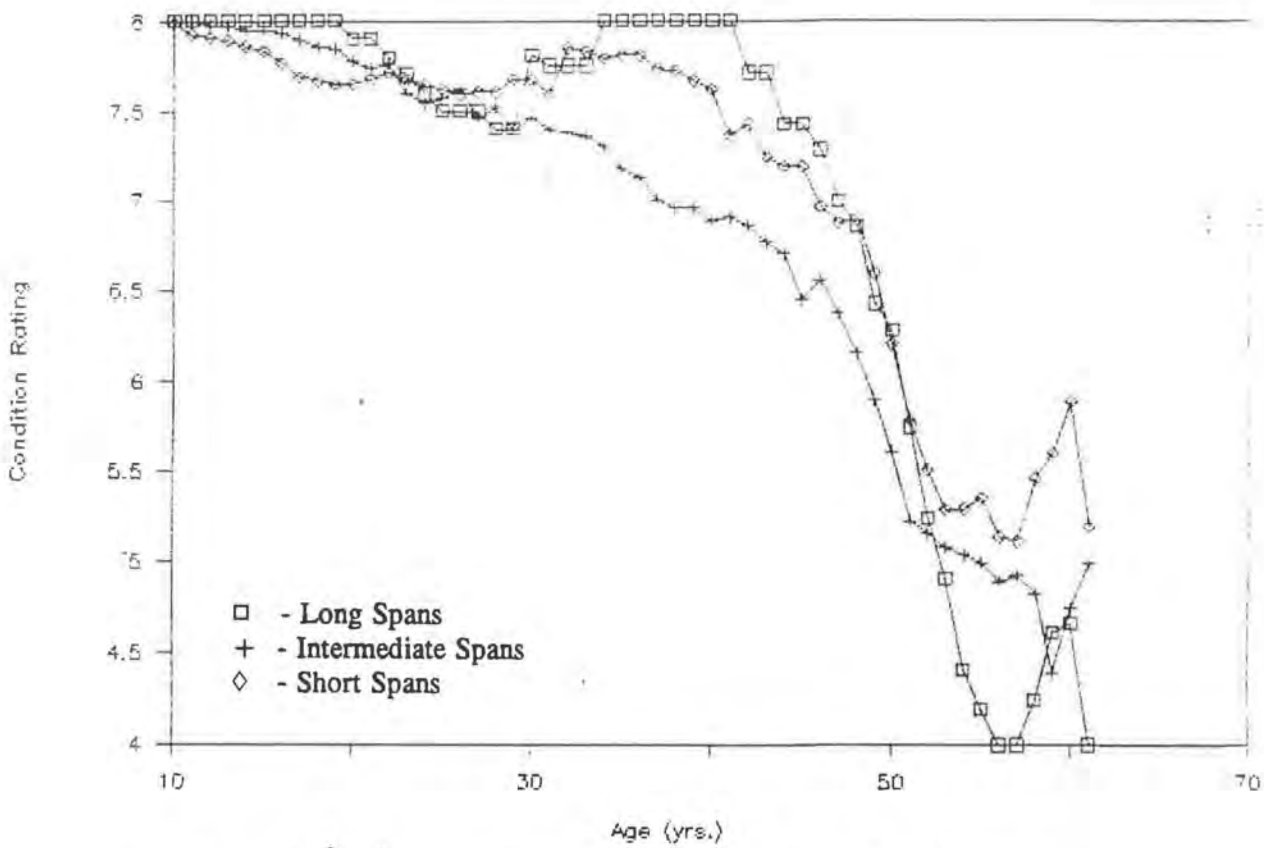


Figure 5.44 Superstructure Condition Rating vs Age Curves for Short, Intermediate and Long Span Bridges

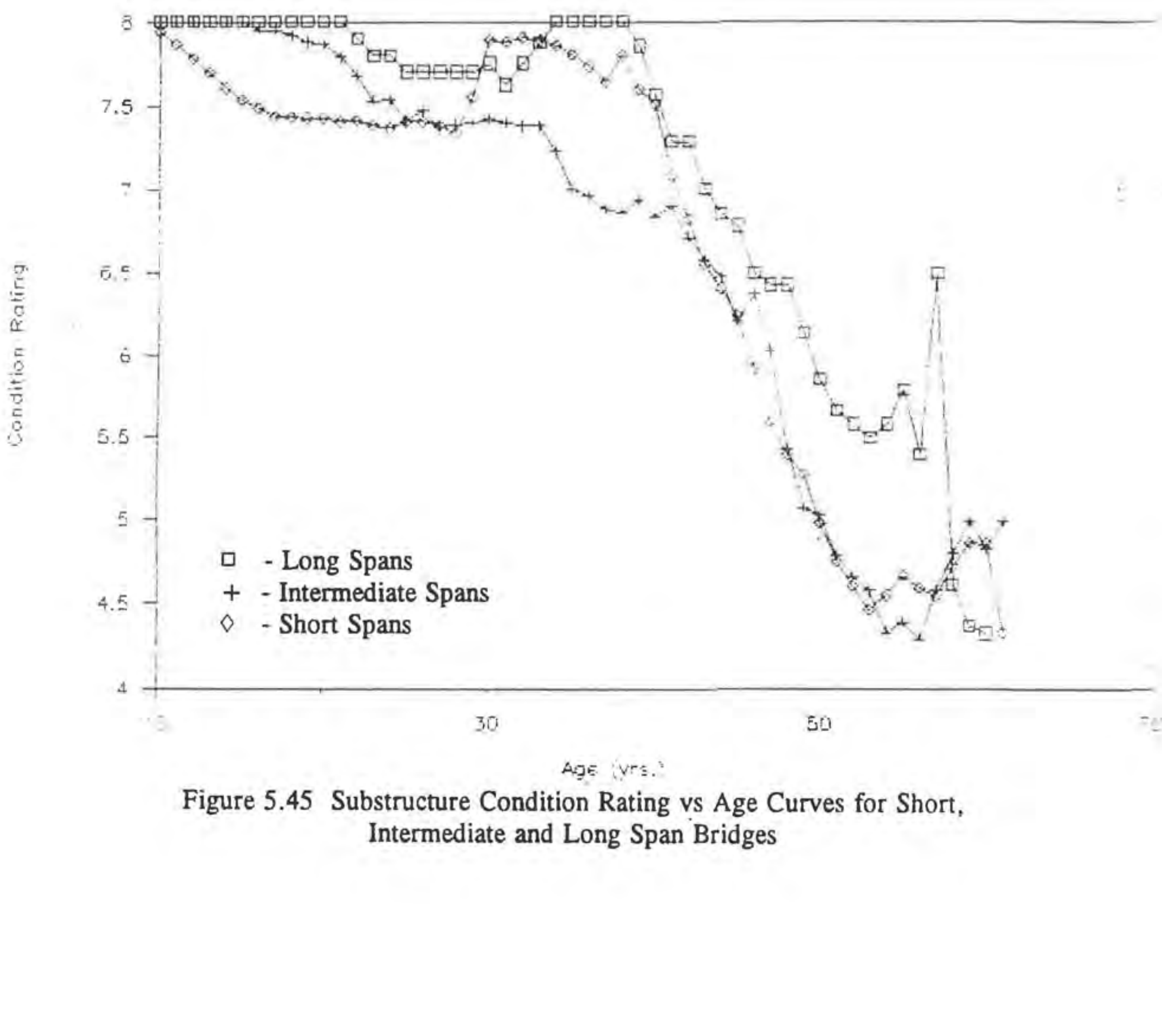


Figure 5.45 Substructure Condition Rating vs Age Curves for Short, Intermediate and Long Span Bridges

5.8.3 Substructures

Bridge substructure durability performances with changing span length were just the reverse of the superstructures. That is, the substructures performed the worst of the 3 major components for short span bridges and the best for long spans. Because of this and the fact that the deck and superstructure durability performances were just the reverse, a statement regarding the best span lengths to use to enhance bridge durability cannot be made. However, the data does indicate that for short bridges, additional attention to enhancing the durability of the bridge substructure should translate into additional bridge durability. And, for long bridges, additional attention to enhancing the durability of the bridge deck and superstructure should result in additional bridge durability.

5.9 Summary and Closure

There are two different modes of failures for bridges, i.e., structural deficient or functionally obsolete. After reviewing ALDOT's historical records of the bridges that are given in AU Subset 143, it appears that the most structural deficient components of this set were the deck (which was the worst component), and the substructure. The worst subcomponents were:

- Wearing surface
- Structure of the deck
- Curbs
- Expansion joints
- Paint
- Deflection under loads
- Wings of the abutment
- Backwall of the abutment.

The major reason for the deck to fail functionally is because of deck geometry. Also, the data showed that the higher the ADT's and ADTT's were, more deterioration was detected. For example, the arterial bridges, subjected to high traffic volumes, were found to have more deterioration than other types. These observations acknowledge key points that, with special attention in the different phases of the design, will enhance the life of the bridge.

The simple span construction is far superior to the continuous span construction for enhancing durability according to the data that was available. The simple span excelled in every component and subcomponent except for the curbs, columns of the bents or piers, and the alignment of members. This gives a good indication of the usefulness of the simple span construction to extend the life of a bridge.

After examining the different lengths of spans, it was concluded that the longer the span was, the more the deterioration was for the deck and superstructure. This indicates that consideration of designing shorter spans would increases the life of the components of the deck and superstructure. It also indicates that more thought to the design of these components for longer spans are needed to enhance the durability of these bridges.

6. RESULTS OF PERSONAL INTERVIEWS AND ALDOT BRIDGE SITE VISITS

6.1 General

In May 1994 Mr. Terry McDuffie, ALDOT Bridge Maintenance Engineer and Mr. Bob Cambell, ALDOT Bridge Testing Engineer led an AU research team on a site visit of approximately 15 bridges within a 40 mile radius of ALDOTs main office in Montgomery, AL. The purpose of the visits was to get a first hand and detailed view of the common types of bridge damage/deterioration in the ALDOT system and their typical methods of repair. Additionally, it allowed comprehensive interviews with two of ALDOTs most knowledgeable bridge performance personnel. The results of the interviews and site visits are presented in the next section. It should be noted that most of the illustrating photos shown in Section 6.2 were taken from Reference [16].

6.2 Results of Site Visits and Interviews

Based on discussions with Mr.'s McDuffie and Cambell, the 10 major causes of and/or bridge deterioration problems in Alabama are felt to be as follows:

1. Faulty deck joints
2. Improper drainage
3. Deck cracking and spalling
4. Concrete cracking and spalling
5. Improper bearing performance
6. Rusting/corroding of structural steel

7. Fatigue cracking at steel girder to diaphram connections and at web stiffeners

8. Abutment erosion

9. Foundation scour

10. Poor quality construction

Brief discussions and/or illustrating photos of these most common deteriorations are presented below. It should be noted that no attempt was made to order this listing based on frequency of occurrence or importance.

6.2.1 Faulty Joints

The common and damaging problem with deck joints is they have a tendency to accumulate dirt, rocks and other roadway debris and become clogged. Then when temperatures increase and expansions try to take place large compressive stresses build up which may crush the girder ends, damage the abutment headwalls and/or damage the approach slab. For bridges on a skew, it causes horizontal misalignment and associated problems.

The reverse of clogging also creates bridge durability problems, i.e., open joints shedding water onto the superstructure and substructure below causing degradation and premature failure of these components. The ideal of course is a properly sealed joint which allows expansion and contractions and prevents clogging and is water tight.

Another common problem is loosening and pop-up of angle irons at angle joints. This problem is caused by the many impact loadings from vehicle traffic. It must be addressed quickly as it is a safety hazard to the travelling public and particularly dangerous to motorcyclists. Photos illustrating some deck joints in badly deteriorated states are shown in Figs. 6.1 and 6.2.

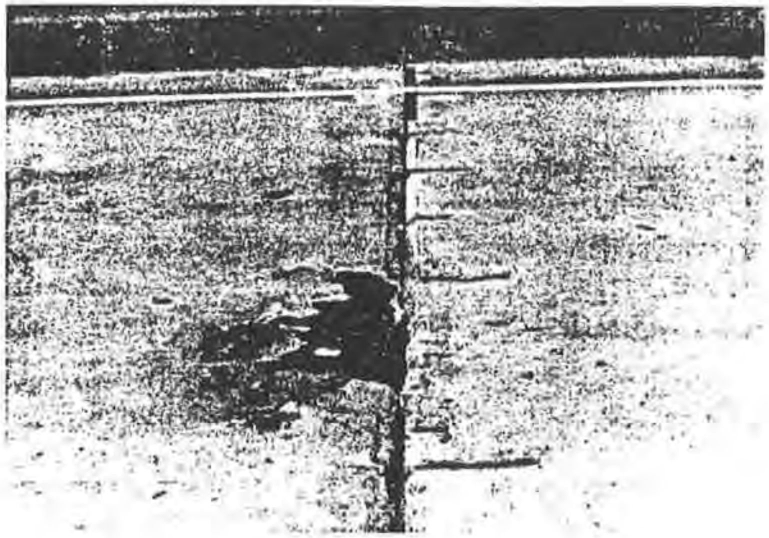


Figure 6.1 Angle Joint with Portions of Angle Missing and Local Concrete Cracking and Spalling [16].

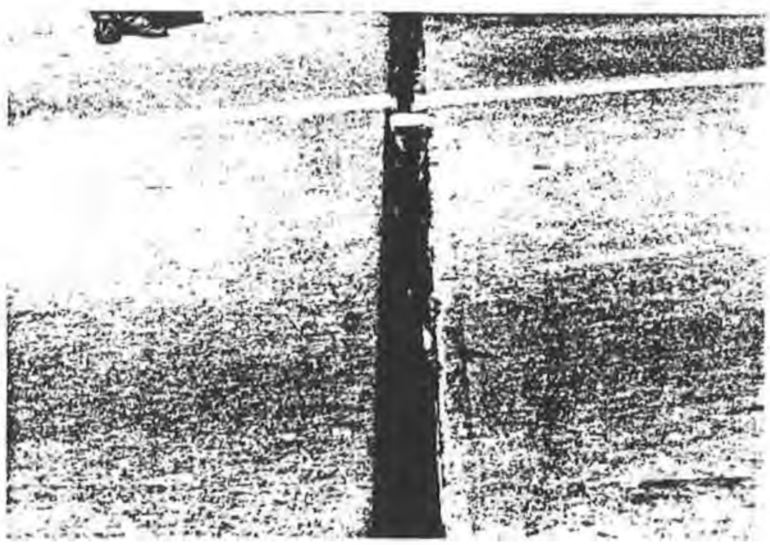


Figure 6.2 Sealed Armor Joint with Advanced Deterioration of Seal Due to the Excessive Opening of Joint [16].

6.2.2 Improper Drainage

The common causes of inadequate deck drainage are

- Small inaccuracies in the deck finish
- Shallow slopes and crowns
- Improper positioning of collection devices
- Existence of too few or too many collection devices
- Accumulation of debris in drainage system (see Figs. 6.3 and 6.4)
- Improper drainage discharge

When drainage runoff is emptied onto elements of the superstructure and substructure significant damages may result. In many bridge decks, water is collected and discharged freely into the air below. When blown by winds the flow of this discharge is redirected and in many cases comes in contact with structural elements of the super and substructures. Drainage of this sort leads to standing water on elements directly beneath the deck such as bent caps, abutment seats, lower flanges of girders, etc. which are shaded from the sun. This standing water leads to scaling, delaminations, and spalling. The photos of Figs. 6-5-6.7 illustrate some of these deleterious effects.

Drainage within the deck is not the only concern when dealing with stormwater runoff. Special attention should also be given to the roadway shoulders at the bridges abutments. If grass in such areas is thick, debris tends to accumulate which directs stormwater flow onto the deck surface instead of below the bridge as intended.

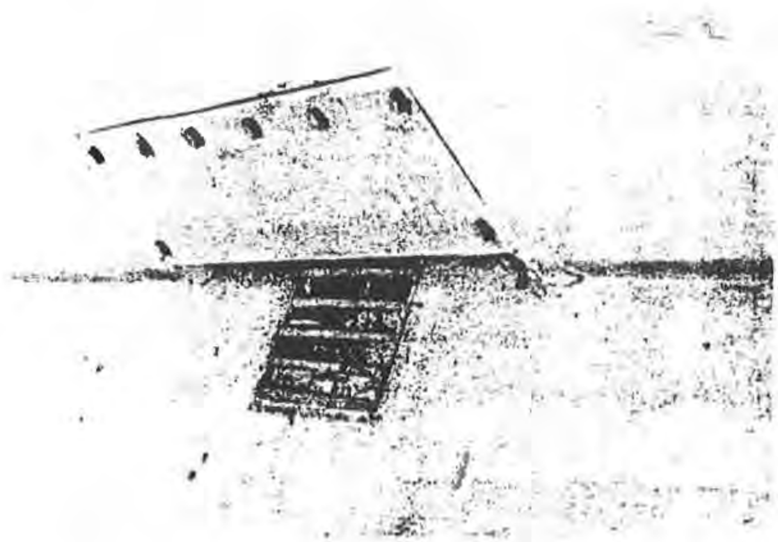


Figure 6.3 Scupper Drain Partially Stopped Up [16].

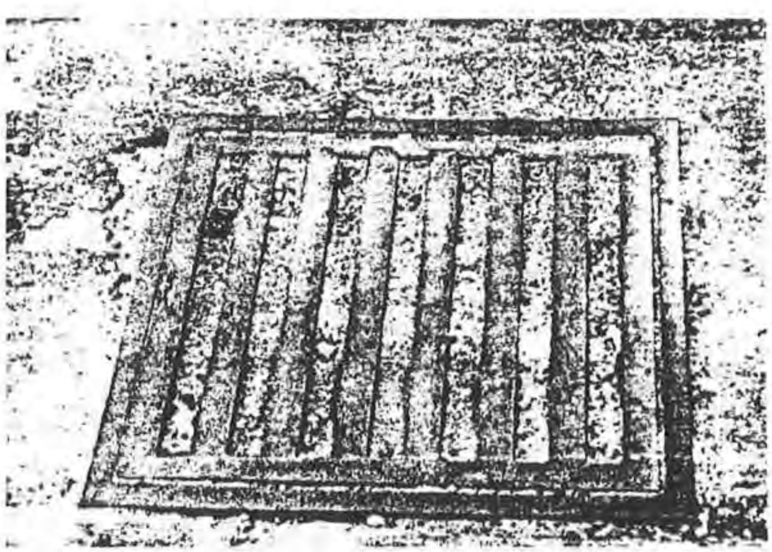


Figure 6.4 Stopped Up Drain [16].

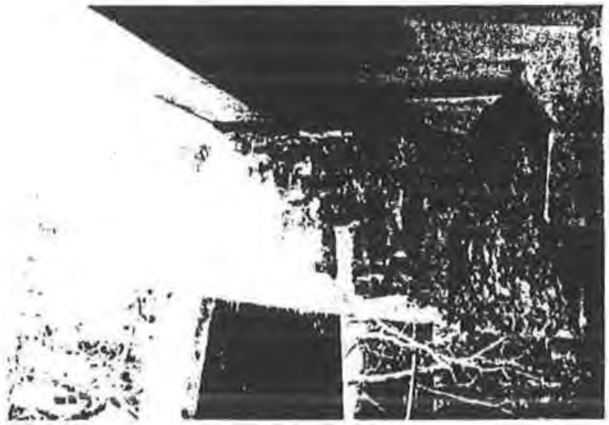


Figure 6.5 Concrete Cap with Serious Spalling and Section Loss [16].

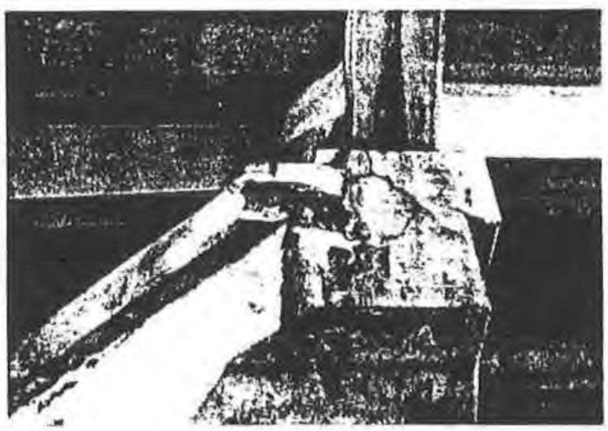


Figure 6.6 Concrete Cracks and Spalling at Cap Ends [16].

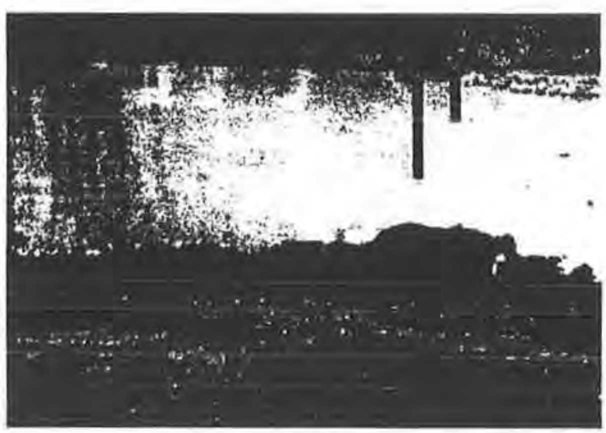


Figure 6.7 Rust on Steel Girder Lower Flange/Web [16].

6.2.3 Deck Cracking and Spalling

Deck cracking due to shrinkage, creep, temperature variations and vehicular loadings will always occur in concrete decks. It is very important in achieving durable decks that the cracking be held to minimal numbers and crack widths. Each crack is a potential source of entry for water and chemicals to attack and corrode the deck reinforcement. This in turn can lead to surface spalling, which leads to greater vehicular impact loadings and more cracks and thus the deterioration rate increases. Figures 6.8 and 6.9 show examples of deck cracking and spalling due to rebar corrosion respectively.

6.2.4 Concrete Cracking and Spalling

Concrete cracking, rebar corrosion, and spalling is not a problem which is limited to decks. All components of all concrete bridges experience some degree of these deteriorations. Holding the degrees to a low level is critical to durability. It is a problem with prestressed concrete bridges as well. Many of the bridges visited had girder and bent cap cracks and spalls due to rebar corrosion as illustrated in Figures 6.10-6.12.

6.2.5 Improper Bearing Performances

Most superstructure damages result from deck joint clogging or leaking, and from poor bearing performances. Figure 6.13 shows a neoprene bearing pad in poor condition. For concrete bridges, the most common superstructure damages are cracked diaphrams and sheared girder ends and sheared bent caps. Cracked diaphrams do not affect the integrity of the structure and are more of a moisture entry concern. However, sheared girder ends and bent caps result in section losses, exposed rebar and prestressing strands, and loss of bearing surface.

Figure 6.8 Concrete Deck with Transverse Cracks [16].

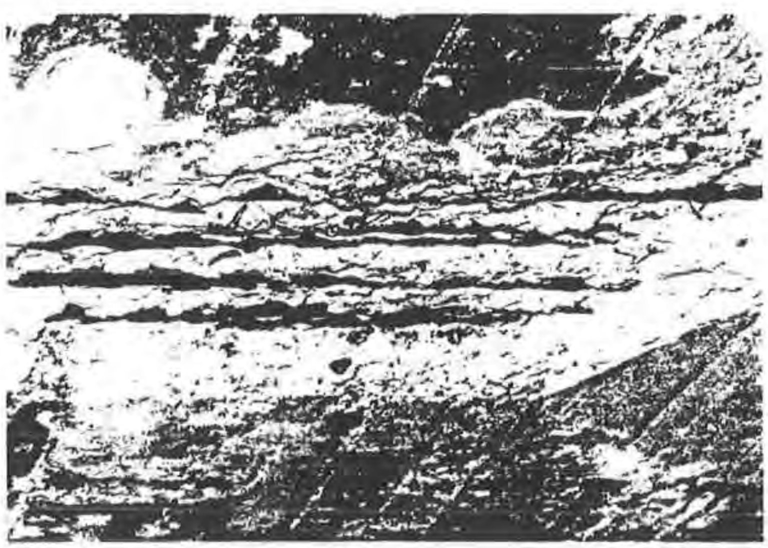


Figure 6.9 Bottom of Concrete Deck with Serious Spalling and Exposed Rebars [16].

Figure 6.10 Concrete Girder with Considerable Spalling and Rebars Exposed [16].

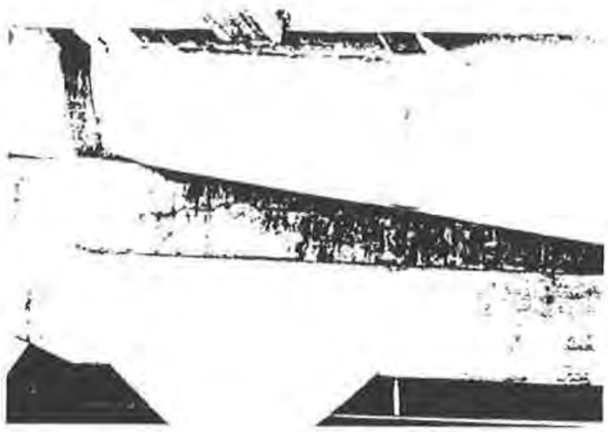


Figure 6.11 Cracks in Concrete Bent Cap.

Figure 6.12 Concrete Cap with Horizontal Cracking.

Sheared girder ends and bent caps result from the freezing of bearings and/or the accumulation of debris in expansion joints. When bearings become frozen and expansion occurs, stresses begin to form at girder ends and bent caps. Likewise, if expansion is limited due to clogged joints, girders will try to expand at their fixed ends causing the formation of shear stresses. Both situations lead to cracking and loss of section and shear capacity as illustrated in Fig 6.14. Photos showing apparent frozen bearings and resulting sheared end girder and bent caps are presented in Figs. 6.15 and 6.16 respectively.

6.2.6 Rusting/Corroding of Structural Steel

Rusting and corrosion of structural steel is one of the primary modes of deterioration for steel superstructure bridges. The northern state de-icing environment is quite severe in this regard. Even in Alabama, due to our humid conditions, relatively high annual rainfall, and salt water condition along the Gulf coast, the corrosion environment for steel can be rather severe. Figures 6.17 and 6.18 show examples of significant rusting on a steel stringer bridge, and severe corrosion at the base of a steel column respectively.

6.2.7 Fatigue Cracking

Fatigue failures a steel diaphram to girder intersections is a major problem with steel bridges. Also fatigue problems at other locations such as stiffener to web welds as shown in Fig. 6.19 are common problems with steel.

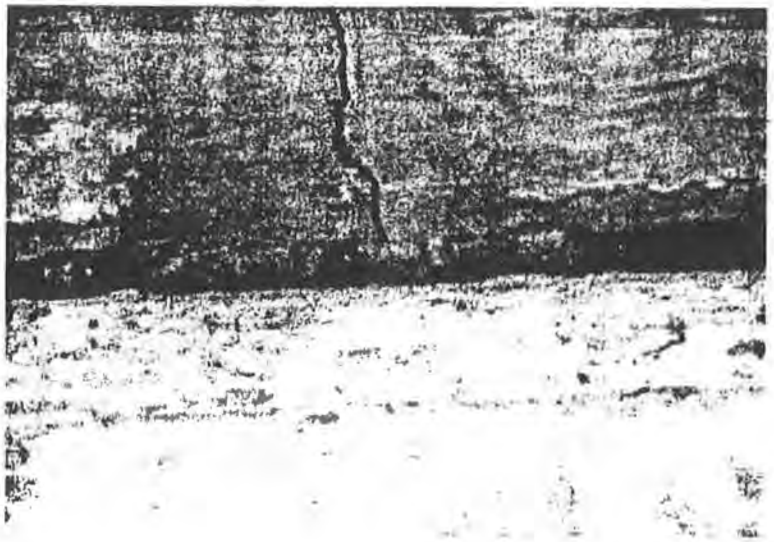


Figure 6.13 Neoprene Bearing Pad Excessively Distorted and in Poor Condition [16].

Figure 6.14 Sketch Illustrating Girder End and Bent Cap Damage Due to Frozen Bearings.

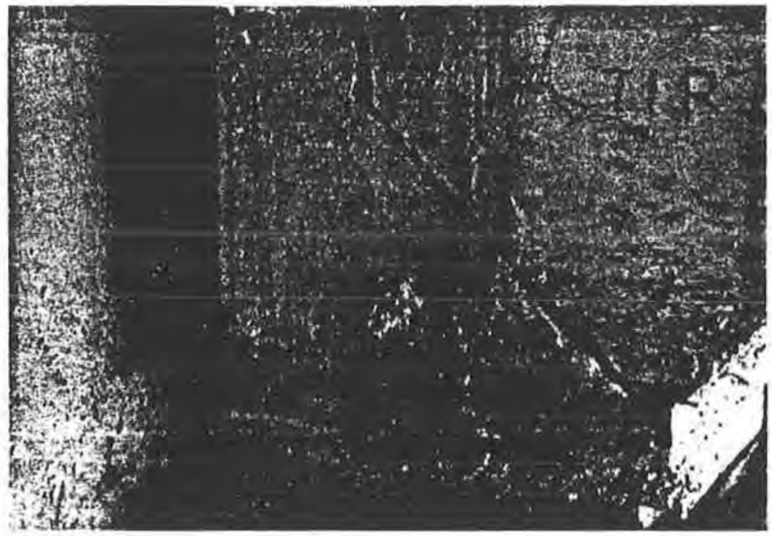


Figure 6.15 Diagonal Girder End Crack Due to Frozen Bearing [16].

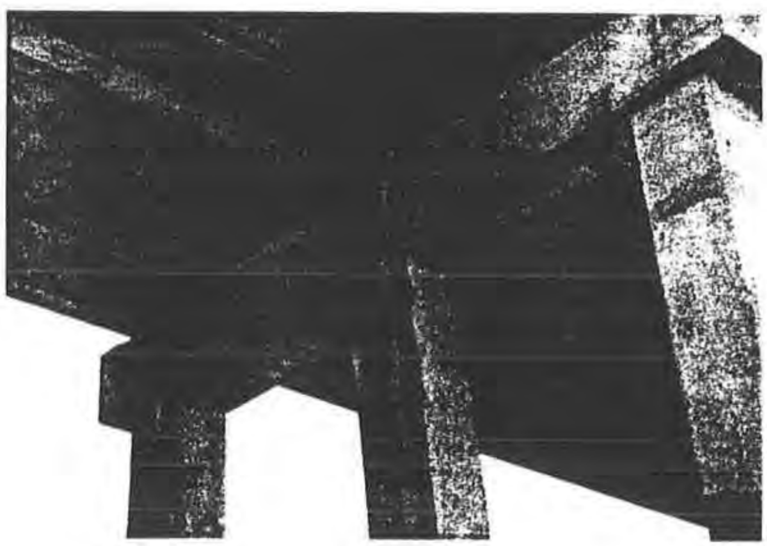


Figure 6.16 Spalling at Bent Cap [16].

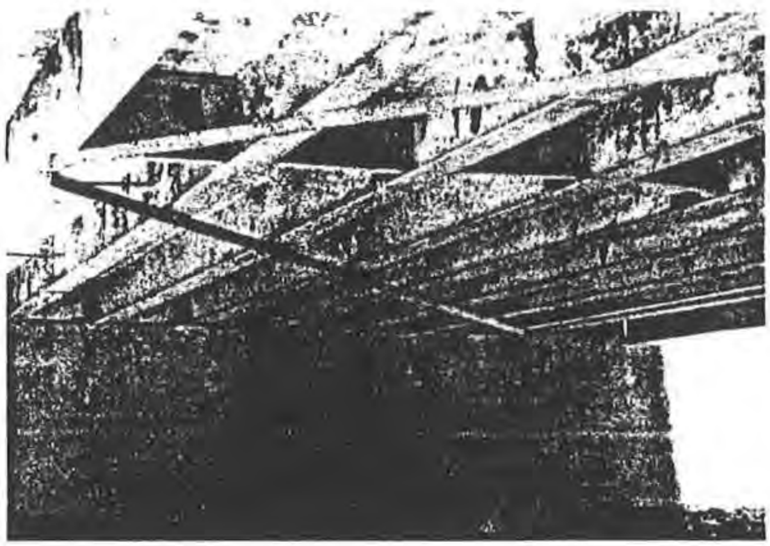


Figure 6.17 Rusting on Steel Stringer Bridge [16].

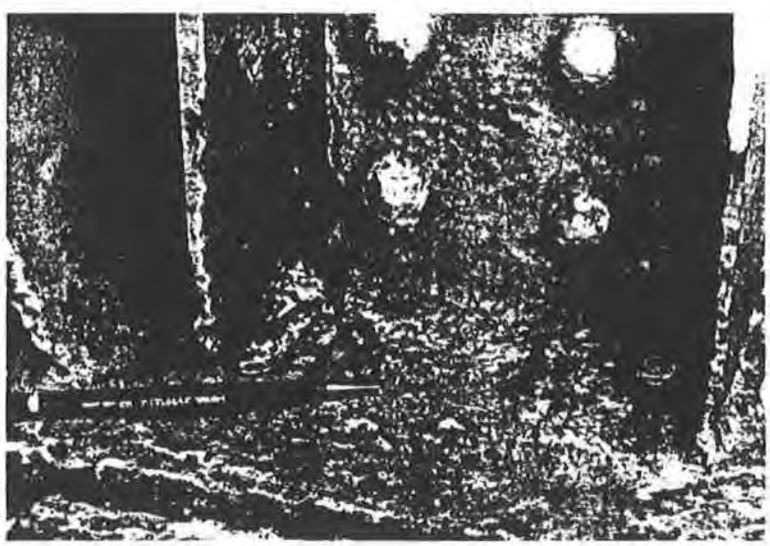


Figure 6.18 Severe Corrosion of Steel Column [16].

6.2.8 Abutment Erosion

Stream bed meandering and instability sometimes causes abutment erosion as illustrated in Fig. 6.20. Also, surface storm drainage sometimes causes erosion near the top of abutment. One new bridge visited had major collapses of slope protection cover slabs like those shown in Fig. 6.20 except at the top of the fill near the abutment seat. The collapses were caused by storm drainage being deflected down and adjacent to the abutment seat.

6.2.9 Foundation Scour

Scour of the substructure support is one of the more frequent factors that cause or lead to foundation distress or structural failure. If the cause of scour can be identified, the repairs made are much more likely to be successful and long lasting. Causes of scour include but are not limited to migration of waterway, inadequate waterway, presence of debris, and removal of upstream bed material such as mining or dredging of sand and gravel. Spur dikes, jetties, deflectors, rip rap, and other solutions may be implemented to protect a bridge's substructure, but if not properly implemented, these solutions can accelerate the original problem or cause new problems.

One of the main difficulties affecting scouring problems is in early detection of the problem. Because of the inaccessibility of most substructures, problems often go unnoticed until severe degrees of scour are reached. Figures 6.21 and 6.22 show examples of scour at bridge bent locations.

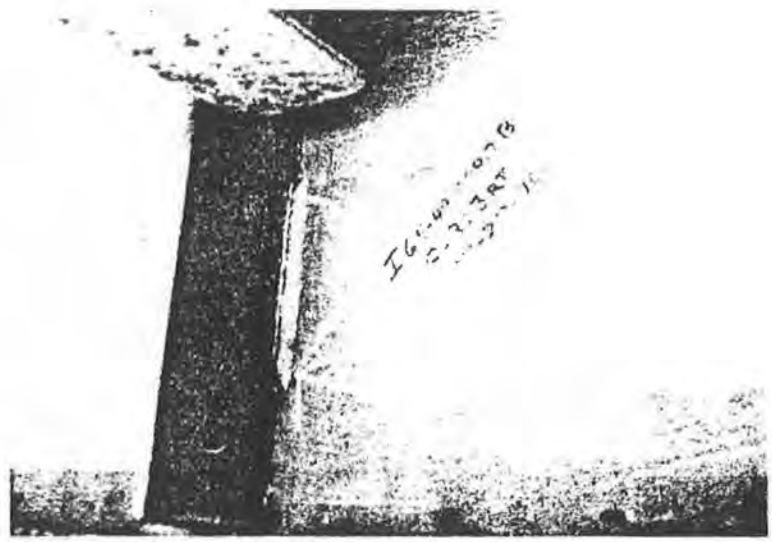


Figure 6.19 Cracked Weld at Location of Web Stiffener [16].

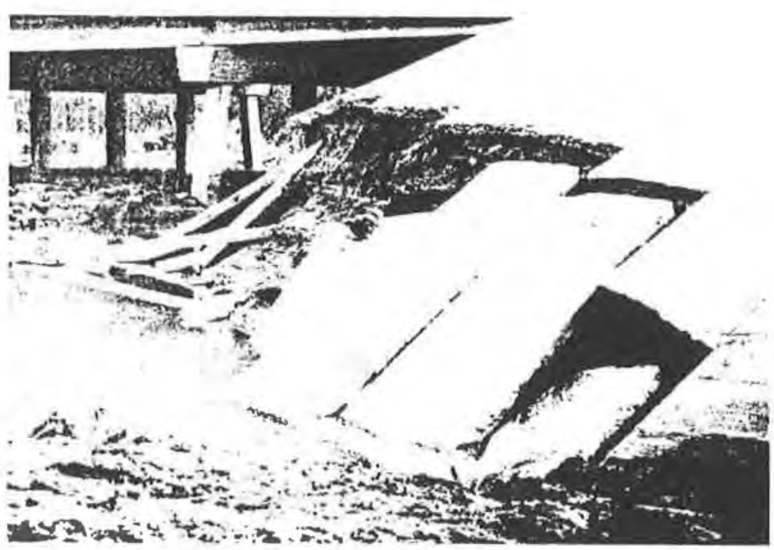


Figure 6.20 Erosion of Abutment Fill [16].

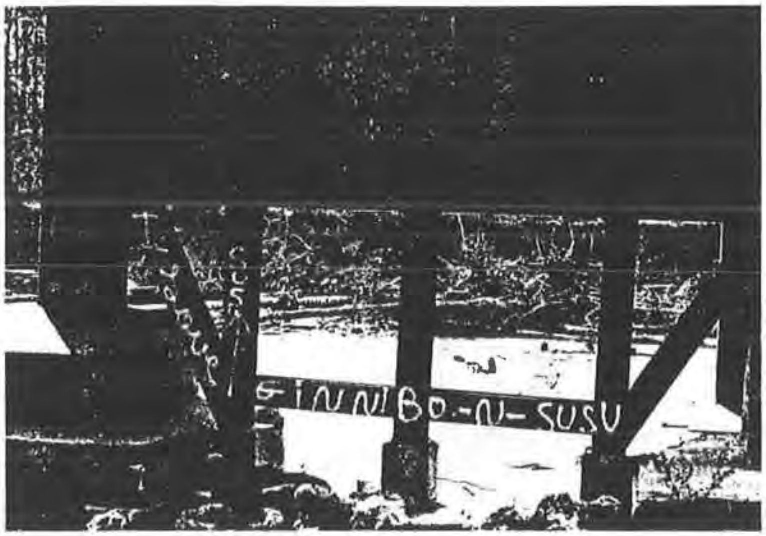


Figure 6.21 Scour at Concrete Spread Footing [16].

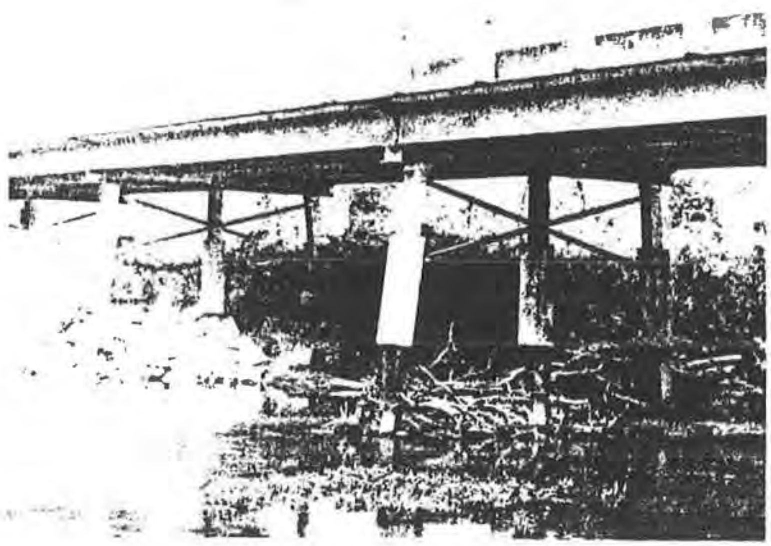


Figure 6.22 Scour at Pile Bents [16].

6.2.10 Poor Quality Construction

In visiting the various bridge sites it was evident that there was a lot of variation in the quality of construction. Small details such as the requirement of placing drain holes in stay-in-place deck forms at construction joints were sometimes overlooked. Such construction oversights do not show up until years later when the deck deteriorates prematurely due to excessive moisture exposure, i.e., what is hurt by this oversight is bridge durability. Other evidence of less than desirable quality of construction observed were misalignment of anchor bolts, misalignment of bent piles and bent columns, and out of plumbness of bent columns. Some of these geometrical errors were not significant to the bridge's strength or serviceability; however, they make one wonder about the accuracy of things like rebar vertical and horizontal placement, concrete cover dimensions, etc. which are critical to bridge strength and serviceability/durability.

It should be noted that it is easy to lay the blame for these shortcomings on the construction contractors. However, inspectors have to take their fair share of the blame. It is the job of the inspector to make sure that all design drawings, specifications and quality control guidelines are met.

7. MAJOR FACTORS AFFECTING BRIDGE DURABILITY/LONGEVITY AND ENHANCEMENT ACTIONS NEEDED

7.1 Major Factors Affecting Durability/Longevity

Based on results of the literature review as presented in Chapter 2, the survey of highway bridge experts in Alabama as presented in Chapter 4, the results of the ALDOT bridge record study as presented in Chapter 5, and the personal interviews and bridge site visits as reported in Chapter 6, the major factors affecting highway bridge durability/longevity performances were identified. These factors, along with comments and actions needed to achieve bridge durability/longevity are as follows:

1. Have a 100-year design life philosophy. Planners, designers, construction, and maintenance personnel should plan, design and work toward 100 year service lives for all permanent highway bridges.

2. Improve projections of traffic and geometry requirements. Inaccuracies in projecting ADT/ADTT, number of lanes required, and lane width requirements seem to be the primary cause of bridge functional obsolence which, in turn, is the primary cause of bridge failures. Growth records and statistics, along with bridge specification changes, should be reviewed and compared with today's conditions to improve the accuracy of future predictions and planning decisions in these areas.

3. Improve projections of waterway parameters. Erosion, scour and flooding are major contributing factors to bridge deterioration. Improvements in projecting requirements to achieve a stable streambed, proper allowances for scour and proper allowances for flood water levels are needed.

4. Adopt an ease of bridge upgrading/rehabilitation strategy. Design bridges to be easily adapted for adding future lanes and/or replacing the bridge deck. Since traffic volumes are constantly increasing, the ability to add new lanes in the future is important to the design. Decks are the worst performing component of a bridge and need replacing during the life of a bridge. The easier to repair or replace a deck, the greater the longevity of the bridge and lower the life-cycle-cost.

5. Design ease of inspection and ease of maintainability into the bridge. This will enhance the quality of inspections and maintenance activities and, in turn, will enhance durability.

6. Make decisions based on life-cycle-cost. Life-cycle-cost should be considered throughout the bridge planning, design, construction, maintenance phases. Total life-cycle cost throughout these phases should be considered in making all decisions.

7. Avoid collision damage. Damages can be caused to a bridge due to collision accidents that occur on or under the bridge. Place protective barricades and guardrails around the substructure and add reflective paint to make them clearly seen at night.

8. Provide sufficient funding. Funding is a problem because, many times, designers will require inferior quality products in order to stay within the budget. If one looks at the life-cycle-cost of the bridge, then it is easily shown that quality products save money. Lowest life-cycle-cost design have a somewhat higher initial cost, and sufficient initial funds are needed to implement this approach.

9. Prohibit the use of studded tires and tire chains. Studded tires and tire chains cause severe local deterioration of bridge decks as well as high surface wear and polishing. Their use should not be allowed in order to maintain deck durability/longevity.

10. Use only quality materials and products. This is especially important at locations of load applications, regions of high cyclic thermal stresses and movements, and at potential locations of water leaks such as adjacent to joints. For concrete components, this will require using quality concrete ingredients.

11. Employ quality concrete mix design. Durability of the concrete mix should be weighed equally with concrete strength. Use low water/cement mixes as they consistently show lower drying shrinkage and creep-strains. This results in lower levels of shrinkage and creep cracking and thus greater durability. Avoid using high cement paste content as it increases the heat of hydration, which, in turn, promotes thermal and shrinkage cracking. Use a concrete with a low-alkali content and clean aggregate. The proper choice of aggregate and cement will alleviate possible reactive chemicals destroying a concrete design. Use air-entrained concrete to enhance freeze-thaw performance and thus durability.

12. Give special consideration to bridge decks and substructures. Bridge decks and substructures are the major components showing the greatest structural failures. Thus, emphasis should be placed on improving the durability performances of these two components by all bureaus and parties involved.

13. Choose appropriate types of material and construction. Choose a material that is ideally suited for the environment in which the bridge will be placed. In choosing the construction type, try to choose a type that can be prefabricated and

easily constructed on-site and that will be sufficiently strong and durable for the anticipated traffic and other loads.

14. Minimize scaling and spalling of the concrete. Concrete mix design, water content, finishing methods, cover, and environmental conditions are primary parameters affecting these factors.

15. Emphasize mitigating concrete cracking in the design phase. Minimizing cracking will reduce water penetrating the concrete and will, in turn, enhance durability. The use of shrinkage-compensating concrete for bridge decks should be explored for this purpose. Also, changes in the deck top mat rebar can help mitigate deck cracking.

16. Mitigate corrosion due to salts and soils. Adding deicing salts to a bridge deck can cause corrosion that will induce cracking of the concrete. Avoid using deicing salts on bridges and roadways leading up to a bridge. Bridges near salt water can also be harmed by the salt's destructive qualities. An appropriate waterproofing system will help mitigate these problems. Also, corrosion can occur if adjacent soil is chemically aggressive to concrete or steel.

17. Avoid rebar corrosion. Use a 2½" concrete cover over the top layer of the deck rebar and at least 1" below the bottom layer of the rebar. Also, use a low water/cement ratio and a low permeability concrete. The use of epoxy coated rebar will be useful in severe climates.

18. Exercise good water control. Good water control is of paramount importance in all phases of a bridge's evolution. For example, water control is needed in:

    - Specifying low water content in concrete mix designs to decrease shrinkage, creep, and permeability
    - Proper curing of concrete
    - Using water sealers and in waterproofing decks
    - Sealing the deck joints
    - Designing deck crown and slope for drainage
    - Designing deck drain/scupper discharge locations
    - Having major components (e.g., abutments) and small details (e.g., stiffner bracket) shed water
    - Streambed stability and scour at foundations
    - Flood water levels and clearances
    - Debris buildup at substructure
    - Storm water and erosion control.

19. Use concrete water sealers. Specify initial use of effective, low-viscosity, concrete deck water sealers, and have the deck periodically resealed by bridge maintenance personnel. Also, specify initial use of effective concrete water sealers (epoxy or silicone sealers) at select critical locations on the bridge, and have these locations periodically resealed by bridge maintenance personnel.

20. Make structural detailing clear and simple to construct. In structural design and detailing, give due consideration to realistic construction tolerances achievable, concrete cover requirements, and other detailing requirements.

21. Design for higher ADTT. High ADT and particularly high ADTT causes an adverse affect on structural deterioration. Designs should be made to mitigate premature deterioration due to high ADTT. Computer analysis methods of today are quite accurate, but, unfortunately, are often perceived as being almost exact and have resulted in the use of lower design factors of safety. Also, the use of higher strength materials has resulted in decreased member sizes and greater deflections and cracking. In the case of concrete, too often, the increased material strength is emphasized to the detriment of its durability. Additionally, heavier trucks running at higher tire pressures are using bridges in greater numbers (some interstate bridges are now recording truck traffic of over 50 percent of the total traffic). All of this results in exceptionally large cyclic loading/stress values, and thus in bridge fatigue. Fatigue loadings and stress evaluations are considered in the design of steel members. However, this is not typically the case for concrete bridges or for concrete component of steel bridges.

22. Select bridge bearings based on durability considerations, and ease of maintenance and replacement. Since bearings are constantly moving, it makes sense that they are a high wear and maintenance intensive component.

23. Minimize the number of deck joints. Expansion, contraction/control, and construction joints are sources of water leakage and deck and superstructure deterioration. They should be eliminated wherever possible. Where used, they should be quality "top-of-the-line" joints. They should also be sealed to prevent water and debris from entering the joint.

24. Use a "belt and suspenders" design philosophy. At select critical locations and/or bridge components, use a "belt and suspenders" design philosophy where redundancy is added into the design. For example, paint weathering steel girders at deck joint locations, so that if the paint fails, the weathering steel will provide the protection.

25. Be innovative and willing to try new materials, design, and construction approaches. For example, Reference [29] states that over 90 percent of the bridges in the U.S. are less than 500 feet long and are candidates for jointless design. However, only a few states use jointless design even though it reduces initial construction costs, significantly reduces joint leakage and water damage problems, and reduces life cycle costs. Another example is that we know prestressing, in general, is very helpful in minimizing cracking and enhancing durability, and that post-tensioned floor systems are widely used in commercial buildings and parking garages. Thus, it would seem that these systems should be excellent candidates for achieving durable bridge decks.

26. Recognize construction and maintenance limitations during design. In the design stage, give due and realistic consideration to construction and maintenance

limitations and variabilities/controllabilities, and adjust the design accordingly. Simplicity and ease of construction will lead to higher quality work and fewer short-cuts or reworks. Designs should be performed for the average contractor and maintenance teams, not the ideal/super group.

27. Recognize and make appropriate design changes to mitigate the primary types of failures for structural steel members. These are:

    - Rusting/corroding at critical location of excessive wetting exposure due to water collection pockets resulting from poor design details.
    - Rusting/corroding with age due to paint deterioration (keep steel painted!)
    - Fatigue at member intersections with diaphragms, stiffners and at bolted and welded connection locations.
    - Brittle fracture (in colder climates)
    - Residual stresses and resulting inferior fatigue strength

28. Prevent corrosion of prestressing tendons. The prestress tendons of a prestressed bridge will corrode if the tendons are not handled carefully. They should not be stored for a long period of time. If they are stored for a short period, then store in a dry climate with rust-prevent oil sprayed on them. Also, allow the diameter of the coil to be large enough to prevent permanent set of the tendons. After the tendons are in place and equilibrium of the forces has been maintained, the tendons need to be grouted.

29. Provide proper bridge drainage. Design drainage outlets to be able to handle the amount of water that will be collected on the bridge. It is recommended to have a few larger drainage outlets rather than many smaller outlets. The water collected should be carried to the ground through a non-leaking pipe system which directs the water away from any component of the bridge, especially the substructure elements.

30. Avoid geometric discontinuities. Abrupt changes in shape and size of different members joining one another can lead to stress concentrations and reduced fatigue strength.

31. Avoid horizontal surfaces where water can stand. One of the most destructive things to components such as bearing devices is standing water. Design platforms to slope away from such devices to keep water from standing and destroying the components. Also, be aware of water trapping devices such as stiffners of steel girders. Water will collect in these places and corrode the steel.

32. Recognize damage affects of shock waves. Shock waves can cause cracking of the concrete piles as they are being driven. Also, sudden movements of the earth as well as vehicular collisions can cause cracking due to shock waves. Awareness and proper design and construction procedures are necessary to mitigate shock wave damages.

33. Employ simple designs. Complex designs normally require a superior contractor to build. For example, a prestressed bridge where there is directional changes in the tendons. This design requires a contractor with special expertise. Simple designs can be handled much easier and with fewer construction problems. Complex designs may be required and/or desired at times. In these cases, the qualified list of contractors eligible for bidding should be more restrictive and only include those having experience in constructing complex designs.

34. Use prefabricated components wherever feasible. There can be better quality control if components are manufactured in a controlled environment and then shipped to the site. Prefabricated components also normally make construction easier and faster.

35. Choose appropriate bridge span/construction type. Bridge deterioration from water leakage at joints is greatest for simple span construction with its short spans and numerous transverse joints. However, bridge deteriorations from excess concrete cracking from traffic and thermal loadings and from fatigue loadings are greater for continuous span construction with its longer spans. Thus, it is not clear whether short simple span construction is better or worse than longer continuous spans from a durability point of view. However, it can be noted that to enhance durability of simple spans, a strong emphasis must be made on keeping all joints clean and sealed! To enhance durability of continuous spans, strong emphasis must be placed on (1) controlling concrete cracking and thus minimizing penetration of water into the bridge, and (2) designing for concrete fatigue.

36. Develop a Bridge Design Bureau Durability Checklist. The Bridge Design Bureau should develop a comprehensive Bridge Durability Checklist and an in-house expert on designing-in bridge durability. This checklist and expert should review all construction documents with a focus on durability. The expert should establish good communications and rapport with corresponding experts in the Construction and Maintenance Bureaus. Whereas, there will be redundancies between the checklist and reviews in the Bridge Design, Construction, Maintenance Bureaus, each expert will be coming from a different background and will be reviewing from a different perspective and the net result of this process should be bridges with excellent durability.

37. Have bridge designs reviewed by construction and maintenance personnel. Early in the design stage, have a knowledgeable representative of the Construction and Maintenance Bureaus review the preliminary construction documents for suggestions and guidance. Later, have these same representatives review the final construction documents prior to going out for bid. This will improve the constructability, quality of construction and later maintainability of the bridge.

38. Develop a Construction and Maintenance Bureau Durability Checklist. The Construction and Maintenance Bureaus should each develop a Bridge Durability Checklist, and an in-house expert at reviewing construction documents to help assure that durability is "designed-in" in all bridge projects.

39. Use only fully qualified construction contractors and subcontractors. Improved methods of getting only quality contractors and workers on the job site need to be explored. For example consideration of life-cycle-cost in some manner should be explored in selecting the construction contractor. In the procedure of selecting the lowest qualified bidder, more emphasis needs to be placed on qualified rather than lowest. In general, the lowest bid and the best quality/durability are not consistent.

40. Improve quality of field concreting operations. Field "manufacturing" of concrete components of bridges affords many opportunities for improper actions that could result in an inferior bridge. Proper concrete mixing, placement, consolidation, finishing and curing are all essential to achieve a quality and durable concrete bridge.

41. Develop concrete deck construction sequences. The construction sequence for concrete decks can significantly affect early deck cracking and long term durability performances. Acceptable sequences for various bridge span types (SS and continuous) should be identified and standardized to the extent possible.

42. Improve concrete formwork design. Improper formwork, or applying loads to the formwork before the concrete has cured, will result in an inferior concrete component.

43. Emphasize quality construction. Quality and details of construction are very important. Small flaws or oversights, many times, will not cause problems until 30-50 years later. In other words, they will not affect initial performance, but will affect durability.

44. Develop and implement appropriate QA/QC processes for bridge construction. Develop clear and concise Guidelines for Quality Assurance Inspection of Bridge Construction.

45. Have high contractor qualification requirements. Develop and implement realistic, measurable, and enforceable qualification requirements for bridge contractors, and certification requirements (for quality concrete construction) for construction foremen and other key personnel.

46. Develop and implement a bridge construction inspector training course. Better prepare and train bridge construction inspectors regarding construction activities significantly impacting concrete quality, in general, and concrete durability, in particular. Develop and implement a training course for ALDOT bridge construction inspectors to better train them on construction activities affecting concrete quality, in general, and concrete durability, in particular. There is a natural tendency for inspectors to forget long-term effects and concentrate on the near-term.

47. Develop and implement training course on bridge durability. Have a construction work force (during construction) and a maintenance work force (once the bridge

is operational) which are well educated in matters pertaining to bridge durability.

48. Have a maintenance attitude of "an ounce of prevention is worth a pound of cure." This requires thinking preventative maintenance. For example,

    - Keep steel bridges painted
    - Keep joints clean of foreign matter and sealed against water leakage
    - Keep concrete sealed and waterproofed at critical locations
    - Make repairs in a timely and quality manner
    - Remove debris from deck
    - Keep bearing devices clean of foreign matter.

49. Conduct a standardized annual bridge preventative maintenance activity on each ALDOT bridge. This annual work should include:

    - Clean and reseal joints where needed
    - Seal significant deck cracks
    - Clear all drainage passages
    - Wash/flush bridge of any deicing salts and dirt and foreign matter
    - Lubricate where needed
    - Spot paint where needed
    - Apply water sealers to concrete at select critical locations
    - Check erosion around base of substructure (where possible from an above water surface inspection).
    - Make a "durability" inspection and take action quickly on all significant deficiencies observed. (This inspection will be much narrower in scope and less comprehensive than the biennial bridge condition inspections.)
    - Other bridge durability work as appropriate.

50. Enhance effectiveness of biennial inspections. Continue to conduct comprehensive biennial bridge condition inspections as currently required, but place a higher priority on addressing deficiencies which have a long term impact, i.e., that impact durability/ longevity. Act quickly on all significant deficiencies identified in the inspections.

51. Give greater emphasis to monitoring and mitigating scour. Give greater attention to substructure debris build-up, foundation scour and scour control systems in maintenance activities. Give high priority to correcting any scour related deficiencies.

52. Maintain clean and sealed joints. Faulty joints restrict the movement of spans which will lead to stress buildup. Expansion joints should be maintained in a continuous state of being sealed so as not to allow debris or water to enter the joint.

53. Make quality and permanent bridge repairs. In making bridge repairs, it is important to (1) use quality repair materials which are compatible with the in-place material, and (2) make quality and durable repairs (this usually takes a little

longer and means maintaining traffic control a little longer, but is it extra time well spent).

54. Consider reflective cracking above concrete deck joints. There is probably no way to prevent these cracks. Hence the best course of action appears to be to saw-cut above the joints to control the crack location, and then to seal the cracks with liquid bitumen.

55. Consider bridge age in maintenance activities. As a bridge grows older, deteriorations occurs at a faster rate than compared to a newer bridge. More frequent inspections should be given to a bridge as it matures. Also, a quicker response from the maintenance personnel to the problems found during the inspection will slow down the rate of deterioration.

56. Better prepare and train bridge maintenance personnel regarding bridge durability issues. Develop and implement a training course for ALDOT maintenance personnel at the "hands-on" level which focuses on bridge durability.

57. Enforce compliance of truck traffic to load and tire pressure limitations. Appropriately located fixed-site weighing scales and tire pressure monitoring along with use of portable weigh-in-motion equipment should be employed to minimize incidences of overloaded trucks on our bridges. This, in turn, will enhance bridge durability/longevity.

7.2 Major Factors by Responsible ALDOT Bureaus and Actions Needed to Enhance Durability/Longevity

In order to address the major factors affecting bridge durability/longevity in a systematic and organized manner, it would be helpful to so organize the factors themselves. Potential categories which could be used to give some organization and structure to the factors are shown in Table 7.1.

Table 7.1
Potential Bridge Categories.

Bridge Phases and ALDOT Bureaus/Decision Makers	Primary Bridge Components	Primary Bridge Materials
Planning & Design	Deck	Reinforced Concrete
Bridge Design	Superstructure	Prestressed Concrete
Construction	Substructure	Steel
Maintenance		

Recognizing that improvements in durability performance will come only from conscious decisions by the appropriate responsible decision makers; it was decided to breakdown the major factors and actions to be taken based on the responsible ALDOT bureau i.e., column one in Table 7.1. This breakdown will assist the ALDOT Planning & Design, Bridge Design, Construction and Maintenance Bureaus in taking actions to enhance the durability/longevity of future bridges. A separate table for each bureau is presented in Tables 7.2-7.5.

Table 7.2
Planning and Design Bureau Decisions/Areas of Responsibility Affecting Bridge Durability/Longevity.

Major Factors	Actions Needed
1. Current design life philosophy is too short	• Adopt a 100-year design life philosophy. • State and publish the 100-year design life. • Have all ALDOT bureaus involved in a bridge's evolution do their work based on a 100-year design life objective.
2. Improve projections of traffic and geometry requirements	• Work with historical growth records for the area in which the bridge will be built in order to project future growth in that area. • Determine future plans of the cities/areas that the bridge will serve. If major city/area growth is expected, then design bridge accordingly. • Study bridge geometry and safety requirement changes over the past 50 years and project future changes in this area. • Use longer (100 years) projection times to be consistent with 100-year design life.
3. Improve projections of waterway parameters	• Same as #2 actions above except for flood levels, scour, streambed stability, flow rates, opening sizes, erosion, and stormwater on decks.
4. Bridge upgrading and rehabilitations are not planned/allowed for in initial designs	• Plan and design bridges to be easily widened for adding lanes. • Plan and design bridges to simplify deck replacements.
5. Access for inspection and maintenance to some components are too difficult	• Design ease of inspection and maintenance into bridges. • Develop a bridge durability inspection/maintenance checklist. • Establish a bridge durability expert in the Maintenance Bureau and have this expert review bridge construction documents early (early enough to incorporate needed changes) in the design phase.
6. Make decisions based on life-cycle-cost	• Life-cycle-cost should be considered in making all decisions throughout the bridge planning, design, construction, and maintenance phases.

7. Avoid collision damage	• Place protective barricades and guardrails around substructure and add reflective paint to make them clearly seen at night. • Maintain proper underneath vertical and horizontal clearances.
8. Insufficient funding	• Provide sufficient funds to implement a lowest life-cycle-cost design. • Provide sufficient funds to act immediately on distresses and deficiencies as they are detected. • Provide sufficient funds to execute permanent and quality repairs using the best quality materials.
9. Deck damage from studded tires and snow chains	• Prohibit use of studded tires and snow chains. • Where safety dictates use of studded tires/chains, use high performance (strength, density, hardness, etc.) concrete for bridge decks.

Table 7.3
Bridge Design Bureau Decisions/Areas of Responsibility Affecting Bridge Durability/Longevity.

Major Factors	Actions Needed
1. Use only quality materials and products	• Recognize that durability begins with specifying quality materials and products. • Recognize that for materials such as concrete which are "manufactured" in the field, this means quality ingredients and quality "manufacturing."
2. Employ quality concrete mix design	• Recognize that a quality concrete mix design requires -quality ingredients -proper ingredient proportions -good workability to assure quality in-place concrete -low water/cement ratio -minimal shrinkage and creep cracking -low porosity and permeability -use of air entrainment -high strength and <u>durability</u> characteristics -compatibility with the environment in which it will be used.
3. Give special consideration to bridge decks and substructures	• Use high strength and high durability concrete in decks. • Explore improved cracking performance of decks via use of shrinkage-compensating concrete, top rebar mat modifications (see #5 below), extra rebar cover and use of epoxy coated rebars. • Modify foundation designs for scour via improving protection systems and making foundations deeper in scour prone streambeds. • Use high strength and high durability concrete and greater rebar protection schemes in wet-dry (including wet splash) zones of substructures.
4. Choice of material and construction type	• Recognize that prestressed concrete, reinforced concrete, steel, simple span, continuous and jointless bridges are all excellent choices for material and construction type for various situations. • Consider material durability and life-cycle-cost in making decisions.

- Recognize that joint leakage is a major source of bridge deterioration.
- Recognize that there is a greater QC/QA in shop/ prefabricated units as well as faster construction time with prefabricated units.
- Adopt more stringent requirements on bridge concrete based on durability characteristics and environmental exposure as indicated in Fig. 7.1.

5. Scaling and spalling of concrete

- Use low water content concrete.
- Explore use of Type K cement to reduce bleed water and reduce scaling as well as shrinkage cracking and spalling.
- Emphasize proper concrete finishing and curing during construction inspection.
- Have good deck drainage system.
- Place top layer transverse steel <u>below</u> longitudinal steel (since most cracks' are transverse and immediately above transverse steel.).
- Increase concrete cover to 2½" in areas experiencing cracking problems and in corrosive aggressive environments.
- Use epoxy coated top mat rebar in marine or other severe environments.

6. Emphasize mitigating concrete cracking in the design phase

- Same as #5 above.
- Use smaller rebar at closer spacing (same A_s) in top mat of bridge decks.
- Use 40 ksi yield strength rebar.
- Minimize skew angle.
- Maintain low span length to depth (l/d) ratio.

7. Mitigate corrosion due to salts and soils

- Avoid using deicing salts on bridges and roadways adjacent to bridges.
- Minimize concrete cracking via design decisions (see #5 and #6 above).
- Maintain good water sealer on concrete and paint on steel.
- Increase rebar cover in severe saltwater environments.
- Use epoxy coated rebar.
- Keep joints sealed.
- Be knowledgeable about chemical aggressiveness of adjacent soils and design concrete mix accordingly.

8. Avoid rebar corrosion

- See #5, #6, #7 above.

9. Exercise good water control

- Recognize that good water control is needed in
  - specifying low water content in concrete mix designs to decrease shrinkage, creep, and permeability
  - proper curing of concrete
  - using water sealers and waterproofing
  - sealing the deck joints
  - designing deck crown and slope for drainage
  - designing deck drain/scupper discharge locations
  - having major components (e.g., abutments) and small details (e.g., stiffner bracket) shed water
  - streambed stability and scour at foundations
  - flood water levels and clearances
  - debris buildup at substructure
  - storm water and erosion control.

10. Use concrete water sealers

- Use an effective low viscosity concrete deck water sealer during construction.
- Periodically reseal with maintenance personnel.
- Use an effective higher viscosity concrete water sealer (epoxy or silicone sealer) at select critical locations (see Figure 7.1) on bridge during construction, and periodically reseal with maintenance personnel.

11. Make structural detailing clear, water shedding and simple to construct

- Give due consideration to:
  - clarity in detail drawing
  - rain exposure and avoidance of collecting water
  - realistic construction tolerances achievable, concrete cover requirements, and other detailing requirements.

12. Design for higher ADTT

- Recognize that impact and fatigue loadings from overloaded trucks are major factors causing premature deteriorations of bridges.
- Recognize that concrete cracking increases and stiffness decreases due to heavy truck traffic and these lead to more rapid environmental deteriorations.
- Include greater considerations and allowances for fatigue in concrete designs.
- Design for higher ADTT than currently doing by using 100 year projections, and design for overloaded trucks.

13. Deterioration and malfunctioning of bridge bearings	• Select bridge bearings based on durability considerations, and ease of maintenance and replacement. • Recognize bearings are a high wear and high maintenance demand component, and design and maintain accordingly.
14. Deck joints	• Recognize that expansion, contraction/control, and construction joints are sources of water leakage and deck and superstructure deterioration. • Minimize the number of deck joints. • Where absolutely necessary to use, use "top-of-the-line" joints. • Keep joints sealed to prevent water and debris from entering. • Explore the use of jointless bridge construction. • Recognize the extremely detrimental affects of joint clogging and joint leakage and that joints are a maintenance intensity component and act according in maintenance activities.
15. Use a "belt and suspenders" design philosophy	• At select critical locations and/or bridge components, add redundancy into the design by adopting a "belt and suspenders" design philosophy, e.g., - paint weathering steel girders adjacent to deck joint locations - maintain concrete water sealer on girder ends and bent cap at deck joint locations, and seal the joints.
16. Hesitancy to try new materials, design, and construction approaches	• Be innovative and willing to try new materials, design, and construction approaches. Worthy candidates for exploration through R&D are: - Jointless bridges - High performance concrete bridges - Shrinkage-compensating concrete bridge decks - Replaceable bridge decks - Prefabricated modular bridges - Poststressed concrete bridge decks - Carbon composites and fiber reinforced concrete bridge decks.
17. Designs with unreasonable construction and/or maintenance capability demands	• Recognize construction and maintenance limitations and design accordingly. • Consider field working and environmental conditions and adjust design accordingly.

- Simplicity and ease of construction lead to higher quality work and final product.
- Repeats and duplication of details and requirements lead to fewer reworks, fewer change orders and a higher quality work and final product.
- Make designs for the average contractor and maintenance teams, not the ideal/super group.

18. Structural steel failures	• Recognize and make appropriate design changes to mitigate the primary types of failures for structural steel members, i.e., - rusting/corroding at critical location of excessive wetting exposure due to water collection pockets resulting from poor design details. - rusting/corroding with age due to paint deterioration (keep steel painted!) - fatigue at member intersections with diaphragms, stiffners and at bolted and welded connection locations. - Brittle fracture (in colder climates). - Residual stresses and resulting inferior fatigue strength.
19. Corrosion of the prestress tendons	• Do not store tendons for a long period of time. • When briefly stored, store in dry location with rust preventative oil sprayed on them. • Store in coils of large diameter sufficient to avoid permanent set. • After tendons are in place, they should be grouted to avoid corrosion. • Seal member ends to avoid water seepage along the tendons. • Maintain the proper concrete cover on the tendons.
20. Poor bridge drainage	• Have good crown on deck roadway and good bridge slope to keep water moving and to efficiently drain the deck during light and heavy rains. • Design drainage outlets to handle the peak flowrate on the bridge. • Use a few larger drainage outlets rather than many smaller outlets because of reduced incidences of clogging. • Move collected water to the ground through a nonleaking pipe system with a minimal number of turns and joints (to minimize clogging).

	• Direct discharged water away from all bridge components.
21. Geometric discontinuities	• Avoid abrupt changes in shape and size of members joining one another as these lead to stress concentrations and reduced fatigue strength.
22. Horizontal surfaces where water can collect and stand	• Recognize that one of the most destructive things to components such as bearing devices is standing water. • Design platforms to slope away from components such as bearings to keep water from standing. • Avoid potential water trapping details such as at steel girder stiffners.
23. Impact and shock waves	• Exercise care in driving concrete piles to minimize concrete cracking from shock waves. • Design ductility into bridge, particularly at connections to guard against earthquakes and collisions. • Maintain good vertical and horizontal clearances and lane widths to minimize collisions. • Use guardrails and barrier protection systems to guard against collision damage.
24. Complex designs and details	• Recognize that complex designs require a superior contractor and a good QC/QA program. • Keep designs and details clean and simple - they are easier to build and inspect and will result in a higher quality product. • Where complex designs are required or desired, recognize that an experienced contractor, with a reputation for quality work, is needed.
25. Poor quality field work	• Use prefabricated components, wherever feasible, in the design. • Keep designs and details simple and easy to inspect and construct. • Improve the number and quality of construction inspectors and the QC/QA program. • Explore and develop an incentative program to reward contractors for quality work and finished products above and beyond the minimum required.
26. Inherriant problems associated with span/construction type	• Recognize that bridge deterioration from water leakage at joints is greatest for simple span construction with its short spans and numerous transverse joints.

	• Recognize that bridge deteriorations from excess concrete cracking from traffic and thermal loadings and from fatigue loadings are greater for continuous span construction with its longer spans. • In SS construction, emphasize keeping all joints clean and sealed. • In continuous construction, emphasize controlling concrete deck cracking and designing for fatigue.
27. Not emphasizing durability during design.	• The Bridge Design Bureau should develop and use a comprehensive Bridge Durability Checklist. • The Bridge Bureau should have an in-house expert on designing-in bridge durability. • The Bridge Bureau should promote the Construction and Maintenance Bureaus having durability checklist and in-house durability experts as well. • Improve communications between Bridge Design, Construction and Maintenance Bureaus and have all bridge designs reviewed by the latter two bureaus with a focus on bridge durability/longevity.

Table 7.4
Construction Bureau Decisions/Areas of Responsibility Affecting Bridge Durability/Longevity.

Major Factors	Actions Needed
1. Insufficient emphasis on durability during construction	• Develop a Construction Bureau Durability Checklist. • Develop a Construction Bureau in-house expert on bridge durability. This person should review all bridge designs to help assure durability is designed-in and can be constructed-in as well. • Better educate and train construction inspectors on durability issues. • Improve QC/QA program.
2. Poorly qualified construction contractors and subcontractors	• Recognize that, in general, the lowest bid and the best quality/durability are not consistent. • Explore improved methods of getting only quality contractors and workers on the job site. • In the procedure of selecting the lowest qualified bidder, emphasis needs to be placed on <u>qualified</u> rather than <u>lowest</u>. • Explore how life-cycle-cost could rationally be considered in the contractor selection process. • Improve QC/QA program and construction inspector training. This would, in time, eliminate unqualified contractors.
3. Poor quality field concreting procedures	• Recognize that proper concreting procedures are of paramount importance to achieving bridge durability. Proper practices are needed in - mixing, placing and maintaining the specified W/C ratio - consolidation - finishing - curing - avoid placing in extreme temperatures (hot or cold) or otherwise poor weather conditions.
4. Insufficient attention to deck pouring sequence	• Recognize that the construction sequence for decks significantly affects early/green concrete deck cracking and durability performance. • Develop ALDOT guidelines on acceptable deck construction sequences for SS and continuous bridges.

5. Insufficient attention to concrete formwork	• Recognize that formwork rigidity is important in avoiding early concrete cracking. • Do not apply loads (other than the concrete) to the formwork until the concrete is cured. • Make sure rebar is properly positioned relative to the forms (both top and bottom mats), and that it is stable and will remain in position during pouring operations. • Make sure drainage holes are provided in SIP forms on both sides of contraction joints (if they are used) and construction joints, to allow passage of deck seepage water.
6. Poor quality construction	• Help educate the construction industry on the importance of quality construction to durability performance. • Actions listed under #3 above. • Develop and implement appropriate QC/QA processes for bridge construction. • Develop some incentatives for contractors to emphasize quality in their work. • Improve education and training of construction inspectors. • Adopt higher standards of qualification for contractors and subcontractors. • Develop good concreting certification requirements (similar to welding requirements) for construction foremen and other key personnel. Require that a certified person is in charge of every concrete placing operation.
7. Poorly trained construction inspectors	• Development and implement a bridge construction inspector training course. • Emphasize construction quality and durability in training course, and offer course to construction industry personnel as well as to ALDOT and county engineer personnel.

Table 7.5
Maintenance Bureau Decisions/Areas of Responsibility Affecting Bridge Durability/Longevity.

Major Factors	Actions Needed
1. Insufficient preventative maintenance	• Have a maintenance attitude of "an ounce of prevention is worth a pound of cure." • Conduct a standardized annual bridge preventative maintenance activity on each ALDOT bridge. This annual work should include: - Clean deck. - Clean and reseal joints where needed. - Seal significant deck cracks. - Clear all drainage passages. - Wash/flush bridge of any deicing salts and dirt and foreign matter. - Clean bearings and lubricate where needed. - Spot paint where needed. - Apply water sealers to concrete at select critical locations. - Check erosion and scour around base of substructure (where possible from an above water surface inspection). - Make a "durability" inspection and take action quickly on all significant deficiencies observed. (This inspection will be much narrower in scope and less comprehensive than the biennial bridge condition inspections.) - Other bridge durability work as appropriate.
2. Enhance effectiveness of biennial inspections	• Place a higher priority on inspecting for, and acting on, deficiencies having only a long term impact, i.e., that only impact durability/longevity. • Act more quickly on deficiencies identified in the inspection.
3. Streambed instability, erosion and scour	• Give greater attention in periodic inspections to substructure and streambed conditions. • Look for debris build-up, and clear. • Give top priority to correcting any scour related deficiencies.
4. Deck joints	• Recognize that deck joints are the location of greatest deck deficiencies.

- Keep joints clean and sealed.
- Keep concrete adjacent to joints sealed with water sealer (deck, girder ends, and bent caps).
- Keep steel girders painted adjacent to deck joints.

5. Bearing devices
   - Recognize that bearing devices are the subcomponent of the superstructure having the greatest deficiencies.
   - Check frequently and keep clean, lubricated, and in proper working order.

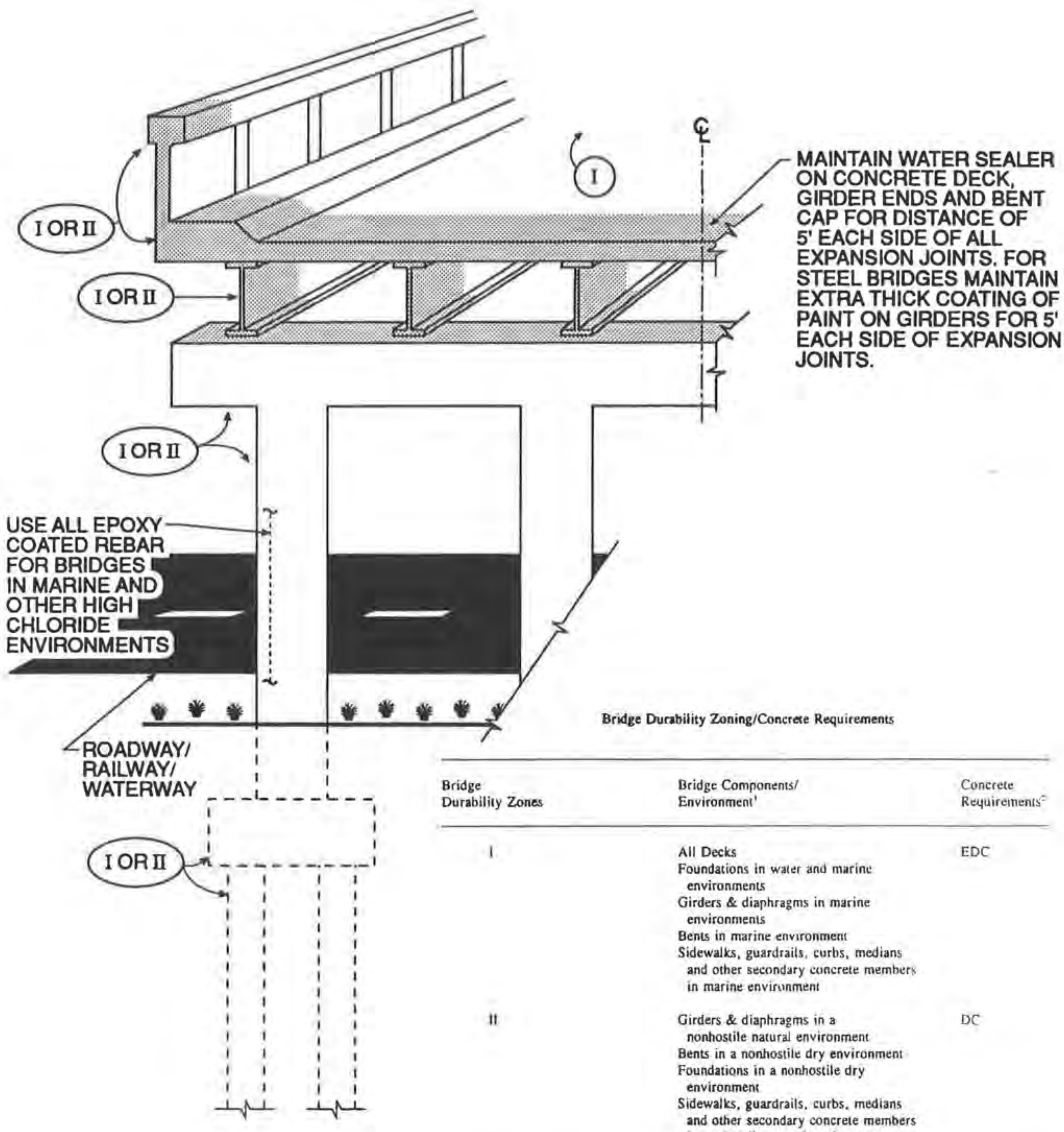
6. Wide variability in quality of repairs
   - Adopt standards for making quality, permanent and durable repairs.
   - Use quality and compatible repair materials.
   - Identify the source of the problem and take actions as necessary to correct.
   - Take extra time to prepare damaged area properly, make repairs and cure (if appropriate) repaired area before opening to traffic.
   - If adjacent traffic makes a quality/durable repair impossible, then take actions to detour traffic so that a permanent and durable repair can be implemented.

7. Reflective cracking in deck surface covering
   - Recognize that it is probably not possible to prevent reflective cracking in deck wearing surface.
   - Saw-cut above joints and significant deck cracks to control crack locations. The locations of where to make saw-cuts needs to be quite accurate to control cracking.
   - Seal saw-cuts with liquid bitumen.

8. More rapid rate of deterioration of older bridges
   - Recognize that once a bridge reaches a state of significant deterioration, it deteriorates at a more rapid rate.
   - Extent the age at which rapid rates of deterioration begin by good preventative maintenance actions as indicated in #1 above.
   - Monitor the condition of older bridges more closely and act quickly on deficiencies as they are identified.

9. Insufficient emphasis on maintenance
   - Develop a Maintenance Bureau Durability Checklist.
   - Develop a Maintenance Bureau in-house expert on bridge durability. This person should review all bridge designs to help assure ease of

	inspection and maintainability is designed-in the bridge. • Develop a training course for ALDOT maintenance personnel which emphasizes preventative maintenance, chronic problem areas, proper repair methods, and other issues related to bridge durability/longevity.
10. Insufficient maintenance funding	• Recognize that each bridge requires annual maintenance expenditures to achieve safety and convenience for the public, and to achieve maximum durability/longevity. • Recognize that older bridges require greater maintenance expenditures than newer bridges. • Develop a rational and accurate annual maintenance expenditure formula for common bridge types to allow accurate estimation of maintenance funding requirements. (This same formula can also be used in estimating life-cycle-costs which can be used in helping to make design decisions and possibly contractor selection decisions.) • Allocate appropriate funding to properly maintain bridges.
11. Overweight trucks and excessive tire pressures	• Strategically locate fixed-site weighing scales throughout the state to enforce weight limitations. • Use portable weigh-in-motion equipment as appropriate to enforce weight limitations. • Check tire pressures on trucks which are in violation of weight limits, and give double citations if necessary.



Bridge Durability Zoning/Concrete Requirements

Bridge Durability Zones	Bridge Components/ Environment[1]	Concrete Requirements[2]
I	All Decks Foundations in water and marine environments Girders & diaphragms in marine environments Bents in marine environment Sidewalks, guardrails, curbs, medians and other secondary concrete members in marine environment	EDC
II	Girders & diaphragms in a nonhostile natural environment Bents in a nonhostile dry environment Foundations in a nonhostile dry environment Sidewalks, guardrails, curbs, medians and other secondary concrete members in nonhostile natural environment	DC

[1]Man imposed bridge environmental factors are not included because ADT and ADTT are the most severe, and they primarily affect only the bridge deck which is already included in Bridge Durability Zone I for all bridges.

[2]DC = durable concrete (concrete with superior durability to that of normal concrete).
EDC = extra durable concrete (concrete with superior durability to that of durable concrete.)

FIGURE 7.1 CONCRETE BRIDGE DURABILITY ZONINGS AND REQUIREMENTS.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The primary factors causing deterioration, or prevention of achieving excellent durability/longevity, of highway bridges throughout the U.S. and actions needed to address these factors were summarized in Chapter 7. Two subsets of those factors which are most applicable and important to bridge deterioration in Alabama, as indicated by a mail questionnaire and personal interviews with bridge and maintenance engineers throughout the state, are as follows:

A. Primary Factors from Mail Questionnaire.

1. Bridge Planning

- Additional planning for streambed stability/scouring.
- Additional planning for foundations and clearances for floods.
- Additional planning for traffic volume, number of lanes, and lane width required.
- Plan and design for longer bridge design life (approximately 100 years).

2. Bridge Structural Design

- Additional attention to substructure design for scour at bridge foundations.
- Design bridge for enhanced future maintainability and inspectability.
- Additional attention to substructure design to resist buildup of debris.
- Design bridge to reduce stress concentrations and fatigue damage at fatigue sensitive locations.
- Projection and selection of more appropriate design traffic loads (design for more cycles of heavier truck loadings).

- Emphasize mitigating concrete cracking.
- Place greater emphasis on improving the performance of bridge substructures.
- Make designs more "friendly" to inspection and maintenance personnel.

3. <u>Bridge Construction</u>

- Emphasize construction quality to have the "as built" bridge match the "as designed" bridge.
- Emphasize on site inspection to ensure conformance to design specifications and quality construction procedures.
- Establish a bridge construction inspection training program for ALDOT personnel.

4. <u>Bridge Maintenance</u>

- Greater attention to periodic routine preventive maintenance activities.
- Act more quickly on problems identified during inspection.
- Keep all bridge joints clean and open.
- Greater attention to maintenance of bridge foundation scour control systems.
- Provide greater funding for bridge maintenance activities.

B. Ten Primary Factors from Personal Interviews.

1. Faulty deck joints
2. Improper drainage
3. Deck cracking and spalling
4. Concrete cracking and spalling
5. Improper bearing performance
6. Rusting/corroding of structural steel
7. Fatigue cracking at steel girder to diaphragm connections and at web stiffners
8. Abutment erosion
9. Foundation scour
10. Poor quality construction

Based on a study of 143 bridges replaced in Alabama during the period 1980-1993, the following observations on durability/longevity were made.

1. Most of the replacements were necessitated because of functional obsolence due to geometry requirements (lane widths were inadequate).

2. The condition rating of most of the bridge components dropped at a rapid rate after about 40 years with projected lives (age when condition rating was projected to reach 4) ranging from around 55-75.

3. Of the 3 major bridge components, decks deteriorated the fastest, followed by the substructure and then the superstructure. However, the durability performance of all 3 major components were quite similar in time history and in total life.

4. Two bridge subcomponents "stick out" as giving excellent durability performances, i.e.,

   - Intermediate bent caps

   - Span supporting girders

5. Three bridge subcomponents "stick out" as giving poor durability performances, i.e.,

   - Joint assemblies

   - Bearing assemblies

   - Abutments

Actions to address these problem areas can be found in Tables 7.2-7.5. Most of the actions are based on current state-of-the-art planning, design, construction and maintenance practices. However, rational new approaches in planning, design, construction and maintenance should be fully explored through R&D efforts to identify new and additional actions and practices to enhance bridge durability/longevity. Some candidates for consideration and study are given in the next section.

In summary, Alabama's bridges have numerous shortcomings which prevent them from achieving excellence in durability/longevity performances; but there are numerous actions which can be taken to correct the shortcomings and achieve excellent longevity. These actions are largely within our control as bridge planners, designers, constructors and maintenance engineers. To a large extent, the future durability/longevity of a bridge is established when the construction documents (bridge plans and specifications) are finalized. As planners and designers we have much more control for designing quality into a bridge than we often recognize. It is true that a poor contractor or poor follow up maintenance activities can have severe negative impacts on bridge durability/longevity. However, the bridge designer still probably has the greatest contribution or noncontribution towards bridge durability. The design can and should be such that the bridge will be robust and not overly demanding of contractor expertise or maintenance personnel. Some recommended actions to enhance durability/longevity are presented in the next section.

8.2 Recommendations

The primary actions recommended to enhance durability/longevity of highway bridges in Alabama are as follows:

- Adopt a 100-year design life philosophy.
- Consider life-cycle-cost in all decisions.
- Use the historical records of changes in bridge safety and geometry requirements and traffic volume and vehicle size changes over the past 50 years to improve the accuracy of projection of future safety, geometric, and traffic load requirements.
- Develop accurate maintenance cost equations and use to allocate necessary funds for proper bridge maintenance.
- Recognize that Planning and Bridge Design Bureaus have primary control of bridge durability.
- In bridge design, give special consideration to:
  - Design ADTT value and be conservative
  - Bridge joint assemblies and joint leakage

- Bearing devices
- Deck and substructure designs
- Streambed stability, erosion and scour control systems
- Structural detailing - keep it clean and simple
- Drainage and stormwater control
- Water sealers
- Select bridge materials and concrete mix designs based on durability and workability as well as strength.

• Develop guidelines to enhance bridge durability/longevity which are appropriate for ALDOT's

  - Planning & Design Bureau
  - Bridge Design Bureau
  - Construction Bureau
  - Maintenance Bureau.

  In the interim, use Tables 7.2-7.5 as these action guidelines.

• Develop a durability expert and checklist in each of the four ALDOT bureaus above to review and make recommendations pertaining to durability early in the design process.

• Develop improved means of achieving quality construction on bridge projects, such as,

  - developing more demanding, but measurable, qualification requirements for contractors
  - incentative programs/contracts for contractors to do quality work
  - improved construction inspections

- more formal and rigorous QC/QA program for bridge construction
- coordinating the development and implementation of a concrete construction certification program, and require that the person in charge of each concrete pour be certified through that program.

- Emphasize preventative maintenance in bridge maintenance activities
- Emphasize monitoring/repairing the primary and chronic problem areas identified in Tables 7.2-7.5 in maintenance activities
- Act quickly on deficiencies as they are detected
- Develop and implement quality bridge durability training courses for bridge planners/designers, construction personnel and inspectors, and maintenance personnel and bridge inspectors.
- R&D new and innovative materials and design/construction approaches to enhance bridge durability/longevity. Worthy candidates for R&D exploration at this time are
  - jointless bridges
  - modular prefabricated bridges
  - replaceable bridge decks
  - poststressed concrete bridge decks
  - shrinkage-compensated concrete decks
  - high performance concrete bridges
  - carbon composites and fiber reinforced decks
- Recognize that to achieve excellence in bridge durability/longevity will require all (or most) of the actions above, rather than just one or two.

REFERENCES

1. AASHTO, "Standard Specifications for Highway Bridges," 15th Edition, AASHTO, Washington, D.C., 1992.

2. ACI, "Building Code Requirements for Reinforced Concrete," ACI 318-89, ACI, Detroit, MI, 1989.

3. ACI Committee 345 Report, "Recommended Practice for Concrete Highway Bridge Deck Construction," ACI Journal, June 1973, pp. 381-415.

4. ACI Committee 363 Report, "State-of-the-Art Report on High-Strength Concrete," ACI 363R-92, ACI, Detroit, MI, 1992.

5. ACI, Durability of Concrete, ACI Publication SP-47, ACI, Detroit, MI, 1975.

6. ACI, "Guide to Durable Concrete," Report by ACI Committee 201-ACI 201.2R-92, ACI, Detroit, MI, April 1992.

7. Amde, M.W.T., Klinger, J.E. and White, E.J., "Performance of Jointless Bridges," Jour. of Performance of Constructed Facilities, Vol. 2, No. 2, May 1988, p. 111-125.

8. ASCE, "Quality in the Constructed Project," Manuals and Reports on Engineering Practice No. 73, ASCE, Vol. 1, New York, pp. 2-3.

9. Austin, R.B., "Bridge Deck Distress and Repair," Paper presented at International Bridge Conference, Pittsburgh, PA, 1992.

10. Baun, M.D., "Steel Fiber Reinforced Concrete Bridge Deck Overlays-Ohio Department of Transportation's Experimental Usage," Paper No. 930124 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

11. Bethlehem Steel Corp., "A Primer on Brittle Fracture," Booklet 1960-A, Steel Design File, Bethlehem Steel Corp., Bethlehem, Pa.

12. Better Roads, "Practical Ideas for Bridge Design and Engineering," Better Roads, May 1988, p. 19-24.

13. Blodgett, O.W., "Design of Welded Steel Structures," The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio, 1966.

14. Blodgett, O.W., Design of Welded Structures, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio.

15. Brester, B., Lin, T.Y. and Scalzi, J.B., Design of Steel Structures, 2nd Edit., John Wiley & Sons, Inc., New York.

16. Bridge Inspection Manual, State of Alabama Highway Department Maintenance Bureau, University of Alabama, September 1990.

17. Brinckerhoff, P., Bridge Inspection and Rehabilitation, John Wiley & Sons, Inc., New York, NY, 1993.

18. Burke, M.P. Jr., "Integral Bridges: Attributes and Limitations," Paper No. 930104 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

19. Cooperative Study Report Printed by PCA, "Report 3 - Durability of Concrete Bridge Decks," 1967.

20. Cooperative Study Report Printed by PCA, "Report 5 - Durability of Concrete Bridge Decks," 1969.

21. Davis, H.E., Tropell, G.E. and Hauck, G.F.W., The Testing of Engineering Materials, McGraw-Hill, 4th Edition, New York, NY, 1982.

22. Derucker, K.N. and Korfiatis, G.P., Materials for Civil and Highway Engineers, Prentice Hall, 2nd Edition, Englewood Cliffs, New Jersey, 1988.

23. Dunker, K.F. and Rabbat, B.G., "Highway Bridge Type and Performance Patterns," Journal of Performance of Constructed Facilities, Vol. 4, No. 3, ASCE, August 1990, pp. 161-173.

24. Dunker, K.F. and Rabbat, B.G., "Performance of Highway Bridges," Concrete International, August 1990, pp. 40-43.

25. Dunker, K.F. and Rabbat, B.G., "Performance of Prestressed Concrete Highway Bridges in the United States - The First 40 Years," PCI Journal, May-June 1992, pp. 48-64.

26. Dunker, K.F. and Rabbat, B.G., "Why America's Bridges Are Crumbling," Scientific American, March 1993, pp. 66-72.

27. FHWA, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," FHWA Report No. FHWA-ED-89-044, DOT-FHWA, Washington, D.C., December 1988.

28. FHWA Technical Advisory, "Uncoated Weathering Steel in Structures," FHWA, October 1989.

29. Gordon, Stanley, "Durability of Highway Bridges," IABSE Symposium Lisbon 1989, Vol. 1, Lisbon, Portugal, Sept. 6-8, 1989, pp. 19-31.

30. Halkyard, Terry D., "Corrosion and Weathering Steel Bridges," Public Roads, Vol. 53, No. 1, June 1989, p. 12-16.

31. International Association for Bridge and Structural Engineers (IABSE) Symposium Report, "Durability of Structures," Vol. 57/1 and Vol. 57/2 (approximately 280 papers pertaining to durability of structures), Lisbon, Portugal, September 1989.

32. Klieger, Paul, "Some Aspects of Durability and Volume Change of Concrete for Prestressing," Jour. of the PCA Research and Development Laboratories, Vol. 2, No. 3, September 1960, pp. 2-12.

33. Litvan, G. and Bickley, J., "Durability of Parking Structures: Analysis of Field Survey," Concrete Durability, ACI Publication SP-100-76, Vol. 2, ACI, Detroit, MI, 1987, pp. 1503-1525.

34. Logan, D.L., Mechanics of Materials, Harder Collins Publishers, New York, 1991.

35. Manning, David, G., Durability of Prestressed Concrete Highway Structures, National Research Council 140, Washington, D.C., November 1988.

36. McCollum, B.F., "Design and Construction of Conventional Bridge Decks That are Resistant to Spalling," Transportation Research Record 604, TRB, Washington, D.C., 1976, pp. 1-5.

37. McCollum, B.F., "Design and Construction of Conventional Bridge Decks that are Resistant to Spalling," TRB Report 604, TRB, National Research Council, Washington, D.C., 1976, pp. 1-5.

38. Moreno, J., "225 W. Wacker Drive - The State of the Art of High-Strength Concrete in Chicago," Concrete International, January 1990, pp. 35-39.

39. Munse, W.H. and Grover, La Motte, Fatigue of Welded Structures, New York: Welding Research Council, New York, 1964.

40. NCHRP Report 57, "Durability of Concrete Bridge Decks," TRB-NRC.

41. Nickson, R.L. and Veshosky, D., "Decision Criteria For Bridge Selection," Modern Steel Construction, September 1992, pp. 39-44.

42. Pritchard, Brian, Bridge Design for Economy and Durability, Thomas Telford Services Ltd., Heron Quay, London, 1992.

43. Radolli, M. and Green, R., "Thermal Stresses in Concrete Bridge Superstructures Under Summer Conditions," Transportation Research Record 547, TRB, Washington, D.C., 1975, pp. 23-36.

44. Raozon, Frank, "Sawcut and Seal May Prevent Reflective Cracks," Roads & Bridges, Vol. 27, No. 3, March 1989, p. 49-51.

45. Report Prepared by an OECD Scientific Expert Group, "Bridge Management," Road Transportation Research - OCDE/OECD, Paris, France, 1992.

46. Report Prepared by an OECD Scientific Expert Group, "Bridge Rehabilitation and Strengthening," Road Transportation Research - OCDE/OECD, Paris, France, 1983.

47. Report Prepared by an OECD Scientific Expert Group, "Durability of Concrete Road Bridges," Road Transportation Research - OCDE/OECD, Paris, France, 1989.

48. Richardson, J., Turner, D., Estes, R. and Green, S., "Calibrating Alabama's Bridge Deficiency Criteria," Paper No. 930660 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

49. Riley, O., "The Decade of the Deck," Journal of the ACI, Vol. 73, 1975, pp. 704-707.

50. Robison, Rita, "Weathering Steel: Industry's Stepchild," Civil Engineering, October 1988, p. 42-45.

51. Rolfe, S.T., "Fracture and Fatigue Control in Steel Structures," Engineering Journal, AISC, 14, 1 (First Quarter 1977), pp. 2-15.

52. Russell, H.G., "High-Strength Concrete," Concrete International: Design & Construction, July 1990.

53. Salmon, C.G. and Johnson, J.E., Steel Structures Design and Behavior, 2nd Edit., Harper & Row Publishers, New York.

54. Scanlon, J.M., "Admixtures - What's New on the Market," Concrete International, October 1992, pp. 28-31.

55. Scanlon, J.M., "Concrete Problems Associated with Hot Climates," Paper presented at 1992 ACI Fall Convention, ACI, San Juan, Puerto Rico, October 1992.

56. Sotiropoulos, S.N. and GangaRao, H.V.S., "Design Anomalies in Concrete Deck - Steel Stringer Bridges," Paper No. 930589 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

57. Takewaka, K. and Mastumoto, "Quality and Cover Thickness of Concrete Based on the Estimation of Chloride Penetration in Marine Environments," Concrete in Marine Environment, ACI Publication SP-109-17, ACI, Detroit, MI, 1988, pp. 381-400.

58. Tork, Andres, "Rehabilitation of Prestressed Concrete Boxed Beam Deck of an Elevated Expressway," Canadian Journal of Civil Engineering, Vol. 16, 1989, p. 8-15.

59. Turner, D.S., Richardson, J.A. and Wong, K.S., "Background Study and Preliminary Design of the ABIMS," Interim Report No. 1, Bureau of Engineering Report 523-39, University of Alabama, March 1991.

60. Turner, D.S., Richardson, J.A. and Wong, K.S., "Development of Bridge Deterioration Models for the ABIMS," Interim Report No. 3, Bureau of Engineering Report 548-39, University of Alabama, December 1991.

61. Van Lund, J.A., "Improving the Quality and Durability of Bridge Modular Expansion Joints," Paper No. 930219 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

62. Yanev, B.S., "Life Cycle Performance of the New York City Bridges," Paper No. 931088 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

63. Yeaton, M. and Moy, J., "The Evolution of Bridge Bearings," Paper No. 931045 presented at 72nd Annual TRB Meeting, Washington, D.C., January 1993.

Appendix A

Direct Durability Requirements

of

AASHTO Standard Specifications for
Highway Bridges - 1992

and

ACI 318-89 Code

and

AISC 1989 Specifications

STANDARD SPECIFICATIONS for HIGHWAY BRIDGES

FIFTEENTH EDITION

1992

CONCRETE

8.16.8.3 Fatigue Stress Limits

The range between a maximum tensile stress and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed:

$$f_f = 21 - 0.33f_{min} + 8(r/h) \qquad (8\text{-}60)$$

where:

- f_f = stress range in kips per square inch;
- f_{min} = algebraic minimum stress level, tension positive, compression negative in kips per square inch;
- r/h = ratio of base radius to height of rolled-on transverse deformations; when the actual value is not known, use 0.3.

Bends in primary reinforcement shall be avoided in regions of high stress range.

Fatigue stress limits need not be considered for concrete deck slabs with primary reinforcement perpendicular to traffic and designed in accordance with the approximate methods given under Article 3.24.3, Case A.

8.22 PROTECTION AGAINST CORROSION

8.22.1 The following minimum concrete cover shall be provided for reinforcement:

	Minimum Cover (inches)
Concrete cast against and permanently exposed to earth	3
Concrete exposed to earth or weather	
Primary reinforcement	2
Stirrups, ties, and spirals	1 1/2
Concrete deck slabs in mild climates	
Top reinforcement	2
Bottom reinforcement	1
Concrete deck slabs which have no positive corrosion protection and are frequently exposed to deicing salts	
Top reinforcement	2 1/2
Bottom reinforcement	1
Concrete not exposed to weather or in contact with ground	
Primary reinforcement	1 1/2
Stirrups, ties and spirals	1
Concrete piles cast against and/or permanently exposed to earth	2

8.22.2 For bundled bars, the minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches, except for concrete cast against and permanently exposed to earth in which case the minimum cover shall be 3 inches.

8.22.3 In corrosive or marine environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, by increasing the denseness and imperviousness to water of the protecting concrete or other means. Other means of positive corrosion protection may consist of, but not be limited to, epoxy-coated bars, special concrete overlays, and impervious membranes; or a combination of these means.*

8.22.4 Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

9.25 COVER AND SPACING OF STEEL

9.25.1 Minimum Cover

The following minimum concrete cover shall be provided for prestressing and conventional steel:

9.25.1.1 Prestressing Steel and Main Reinforcement .... 1 1/2 inch

9.25.1.2 Slab Reinforcement

9.25.1.2.1 Top of Slab .... 1 1/2 inch
When deicers are used .... 2 inch

9.25.1.2.2 Bottom of Slab .... 1 inch

9.25.1.3 Stirrups and Ties .... 1 inch

9.25.1.4 When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.

8.6.5 Special Requirements for Bridge Decks

During periods of low humidity, wind or high temperatures and prior to the application of curing materials, concrete being placed and finished for bridge decks shall be protected from damage due to rapid evaporation. Such protection shall be adequate to prevent premature crusting of the surface or an increase in drying cracking. Such protection shall be provided by raising the humidity of the surrounding air with fog sprayers operated upwind of the deck, the use of wind-breaks or sunshades, additionally reducing of the temperature of the concrete, scheduling placement during the cooler times of days or nights, or any combination thereof.

For bridge decks that are located over or adjacent to salt water or when specified, the maximum temperature of the concrete at time of placement shall be 80°F.

8.6.6 Concrete Exposed to Salt Water

Unless otherwise specifically provided, concrete for structures exposed to salt or brackish water shall be Class S for concrete placed under water and Class A for other work. Such concrete shall be mixed for a period of not less than 2 minutes and the water content of the mixture shall be carefully controlled and regulated so as to produce concrete of maximum impermeability. The concrete shall be thoroughly consolidated as necessary to produce maximum density and a complete lack of rock pockets. Unless otherwise indicated on the plans, the clear distance from the face of the concrete to the reinforcing steel shall be not less than 4 inches. No construction joints shall be formed between levels of extreme low water and extreme high water or the upper limit of wave action as determined by the Engineer. Between these levels the forms shall not be removed, or other means provided, to prevent salt water from coming in direct contact with the concrete for a period of not less than 30 days after placement. Except for the repair of any rock pockets and the plugging of form tie holes, the original surface as the concrete comes from the forms shall be left undisturbed. Special handling shall be provided for precast members to avoid even slight deformation cracks.

8.6.7 Concrete Exposed to Sulfate Soils or Water

When the special provisions identify the area as containing sulfate soils or water, the concrete that will be in contact with such soil or water shall be mixed, placed and protected from contact with soil or water as required for concrete exposed to salt water except that the protection period shall be not less than 72 hours.

8.7 HANDLING AND PLACING CONCRETE

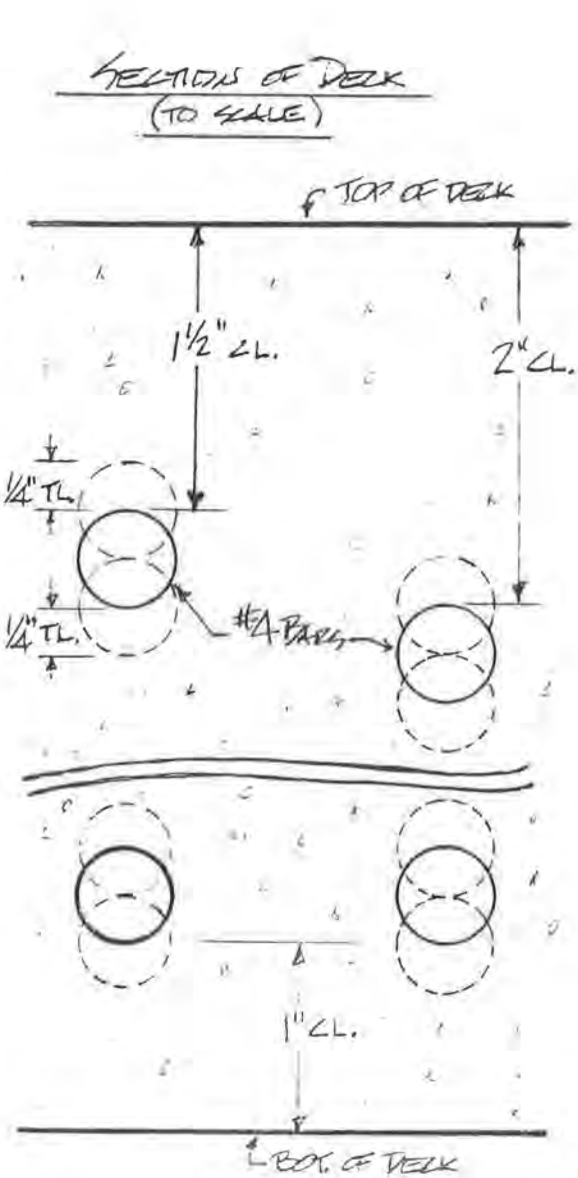
8.7.1 General

Concrete shall be handled, placed and consolidated by methods that will not cause segregation of the mix and will result in a dense homogeneous concrete which is free of voids and rock pockets. The methods used shall not cause displacement of reinforcing steel or other materials to be embedded in the concrete. Concrete shall be placed and consolidated prior to initial set and in no case more than 1-1/2 hours after the cement was added to the mix. Retempering the concrete by adding water to the mix shall not be done.

Concrete shall not be placed until the forms, all materials to be embedded and, for spread footings, the adequacy of the foundation material have been inspected and approved by the Engineer. All mortar from previous placements, debris and foreign material shall be removed from the forms and steel prior to commencing placement. The forms and subgrade shall be thoroughly moistened with water immediately before concrete is placed against them. Temporary form spreader devices may be left in place until concrete placement precludes their need, after which they shall be removed.

Placement of concrete for each section of the structure shall be done continuously without interruption between planned construction or expansion joints. The delivery rate, placing sequence and methods shall be such that fresh concrete is always placed and consolidated against previously placed concrete before initial set has occurred in the previously placed concrete.

During and after placement of concrete, care shall be taken not to injure the concrete or break the bond with reinforcing steel. Workmen shall not walk in fresh concrete. Platforms for workmen and equipment shall not be supported directly on any reinforcing steel. Once the concrete is set, forces shall not be applied to the forms or to reinforcing bars, which project from the concrete, until the concrete is of sufficient strength to resist damage.



9.6 PLACING AND FASTENING

9.6.1 General

Steel reinforcement shall be accurately placed as shown on the plans and firmly held in position during the placing and setting of concrete. Bars shall be tied at all intersections around the perimeter of each mat and at not less than 2 foot centers or at every intersection, whichever is greater, elsewhere. Bundled bars shall be tied together at not more than 6 feet centers. For epoxy-coated reinforcement, tie wire and metal clips shall be plastic or epoxy coated. If fabric reinforcement is shipped in rolls, it shall be straightened into flat sheets before being placed. Welding of cross bars (tack welding) will not be permitted for assembly of reinforcement unless authorized in writing by the Engineer.

9.6.2 Support Systems

Reinforcing steel shall be supported in its proper position by use of mortar blocks, wire bar supports, supplementary bars or other approved devices. Such devices shall be of such height and placed at sufficiently frequent intervals so as to maintain the distance between the reinforcing and the formed surface or the top surface of deck slabs within 1/4 inch of that indicated on the plans.

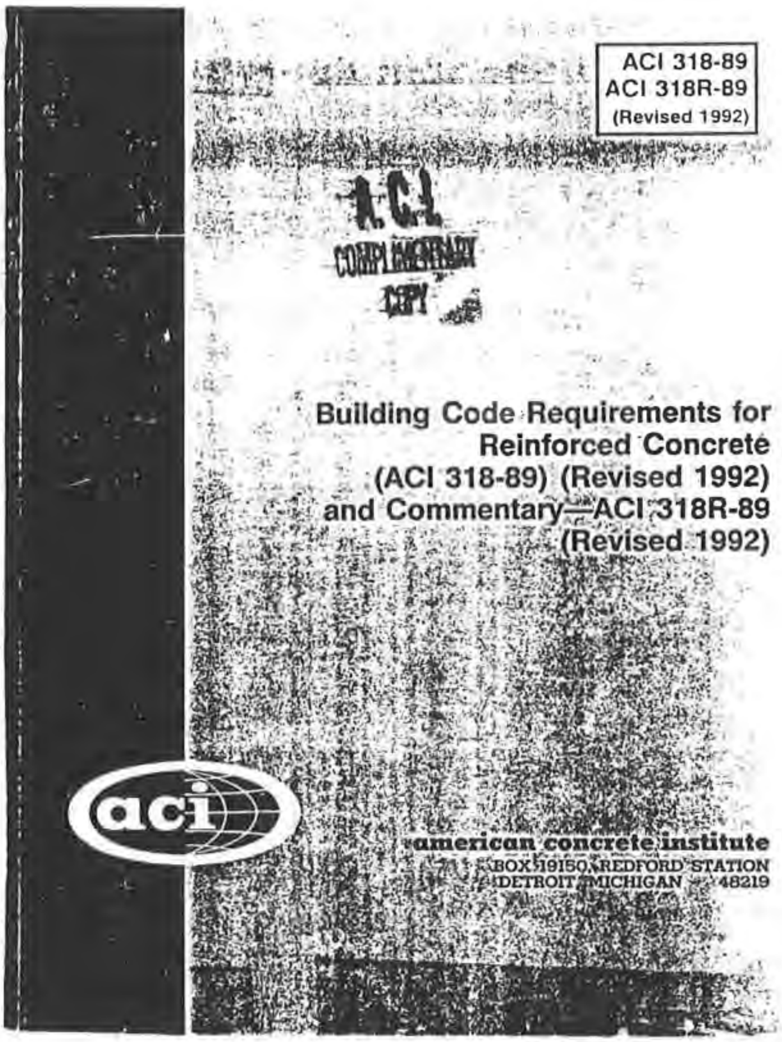
Platforms for the support of men and equipment during concrete placement shall be supported directly on the forms and not on the reinforcing steel.

9.6.3 Mortar Blocks

Mortar blocks shall have a compressive strength not less than that of the concrete in which they are to be embedded. The face of blocks in contact with forms for exposed surfaces shall not exceed 2 inch by 2 inch in size and shall have a color and texture that will match the concrete surface. When used on vertical or sloping surfaces, such blocks shall have an embedded wire for securing the block to the reinforcing. When used in slabs, either such a tie wire or, when the weight of the reinforcing is sufficient to firmly hold the blocks in place, a groove in the top of the block may be used. For epoxy-coated bars, such tie wires shall be plastic or epoxy coated.

9.6.4 Wire Supports

Wire bar supports, such as ferrous metal chairs and bolsters, shall conform to industry practice as described in the *Manual of Standard Practice of the Concrete Reinforcing Steel Institute.* Such chairs or bolsters which bear against the forms for exposed surfaces shall be either Class 1—Maximum Protection (Plastic Protected) or Class 2, Type B-Moderate Protection (Stainless Steel Tipped) for which the stainless steel conforms to ASTM A493, Type 430. For epoxy-coated reinforcement, all wire bar supports and bar clips shall be plastic or epoxy coated.



ACI 318-89
ACI 318R-89
(Revised 1992)

A.C.I.
COMPLIMENTARY
COPY

Building Code Requirements for Reinforced Concrete (ACI 318-89) (Revised 1992) and Commentary—ACI 318R-89 (Revised 1992)

aci

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DETROIT, MICHIGAN 48219

PART 3 – CONSTRUCTION REQUIREMENTS

CHAPTER 4 – DURABILITY REQUIREMENTS

CODE

4.0 – Notation

f'_c = specified compressive strength of concrete, psi

4.1 – Freezing and thawing exposures

4.1.1 – Normal weight and lightweight concrete exposed to freezing and thawing or deicer chemicals shall be air entrained with air content indicated in Table 4.1.1. Tolerance on air content as delivered shall be ± 1.5 percent. For specified compressive strength f'_c greater than 5000 psi, air content indicated in Table 4.1.1 may be reduced 1 percent.

COMMENTARY

Chapters 4 and 5 of earlier editions of the Code have been reformatted to emphasize the importance of considering durability requirements before the designer selects f'_c and cover over the reinforcing steel.

Maximum water-cement ratios of 0.40, 0.45, and 0.50 that may be required for concretes exposed to freezing and thawing, sulfate soils or waters, or for preventing corrosion of reinforcement will typically be equivalent to requiring an f'_c of 4750, 4500, or 4000 psi, respectively. Generally, the required average concrete strengths, f'_{cr}, will be 500 to 700 psi higher than the specified compressive strength, f'_c. Since it is difficult to determine accurately the water-cement ratio of concrete during production, the f'_c specified should be reasonably consistent with the water-cement ratio required for durability. Selection of an f'_c which is consistent with the water-cement ratio selected for durability will help ensure that the required water-cement ratio is actually obtained in the field. Because the usual emphasis on inspection is for strength, test results substantially higher than the specified strength may lead to a lack of concern for quality and production of concrete which exceeds the maximum water-cement ratio. Thus an f'_c of 3000 psi and a maximum water-cement ratio of 0.45 should not be specified for a parking structure, if the structure will be exposed to deicing salts.

The code does not include provisions for especially severe exposures, such as acids or high temperatures, and is not concerned with aesthetic considerations such as surface finishes. These items are beyond the scope of the code and must be covered specifically in the project specifications. Concrete ingredients and proportions must be selected to meet the minimum requirements stated in the code and the additional requirements of the contract documents.

R4.1 – Freezing and thawing exposures

R4.1.1 – A table of required air contents for frost-resistant concrete is included in the code, based on **"Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete"** (ACI 211.1).[4.1] Values are provided for both severe and moderate exposures depending on degree of exposure to moisture or deicing salts. The entrained air will not protect concrete containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition. In Table 4.1.1, a "severe exposure" is one where, in a cold climate the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks,

CODE

TABLE 4.1.1 — TOTAL AIR CONTENT FOR FROST-RESISTANT CONCRETE

Norminal maximum aggregate size, in.*	Air content, percent	
	Severe exposure	Moderate exposure
3/8	7-1/2	6
1/2	7	5-1/2
3/4	6	5
1	6	4-1/2
1-1/2	5-1/2	4-1/2
2†	5	4
3†	4-1/2	3-1/2

*See ASTM C 33 for tolerances on oversize for various nominal maximum size designations.

†These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1-1/2 in. is removed by handpicking or sieving and air content is determined on the minus 1-1/2 in. fraction of mix. (Tolerance on air content as delivered applies to this value.) Air content of total mix is computed from value determined on the minus 1-1/2 in. fraction.

4.1.2 – Concrete that will be subject to freezing and thawing in a moist condition, intended to have low permeability to water or be exposed to deicing salts, brackish water, sea water, or spray from these sources shall conform to the requirements of Table 4.1.2.

TABLE 4.1.2 — REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

Exposure condition	Maximum water-cement ratio, normal weight aggregate concrete	Minimum f'_c, lightweight aggregate concrete
Concrete intended to have low permeability when exposed to water	0.50	3750
Concrete exposed to freezing and thawing in a moist condition	0.45	4250
For corrosion protection for reinforced concrete exposed to deicing salts, brackish water, seawater or spray from these sources	0.40*	4750*

*If minimum concrete cover required by 7.7 is increased by 0.5 in., water-cement ratio may be increased to 0.45 for normal weight concrete, or f'_c reduced to 4250 psi for lightweight concrete.

4.1.3 – The minimum cement content of concrete mixtures exposed to freezing and thawing in the presence of deicing chemicals shall be 520 lb of cement meeting ASTM C 150 or C 595 per cu yd of concrete.

4.1.4 – The water-cement ratio required in Tables 4.1.2 and 4.2.1 shall be calculated using the weight of cement meeting ASTM C 150 or C 595 plus the weight of fly ash or pozzolan meeting ASTM C 618 and/or slag meeting ASTM C 989, if any.

COMMENTARY

sidewalks, parking garages, and water tanks. A "moderate exposure" is one where in a cold climate the concrete will be only occasionally exposed to moisture prior to freezing, and where no deicing salts are used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil. Section 4.1.1 permits 1 percent lower air content for concrete with f'_c greater than 5000 psi. Such high-strength concretes will have lower water-cement ratio and porosity and, therefore, improved frost resistance.

R4.1.2 – The maximum water-cement ratios for normal weight concrete and minimum compressive strengths for lightweight concrete are required to provide adequate durability, corrosion protection, or sulfate resistance. Minimum levels of compressive strength are specified for lightweight concrete (in lieu of water-cement ratios) because determination of the absorption of lightweight aggregates is uncertain, making calculation of water-cement ratio impractical. For lightweight aggregate concrete, a minimum strength level will ensure the use of high quality cement paste.

CODE

4.2 – Sulfate exposures

4.2.1 – Concrete to be exposed to sulfate-containing solutions or soils shall conform to requirements of Table 4.2.1 or be made with a cement that provides sulfate resistance and used in concrete with maximum water-cement ratio or minimum compressive strength from Table 4.2.1.

COMMENTARY

R4.2 – Sulfate exposures

Concrete exposed to injurious concentrations of sulfates from soil and water should be made with a sulfate-resisting cement. Table 4.2.1 lists the appropriate types of cement and the maximum water-cement ratios (or minimum strengths for lightweight concrete) for various exposure conditions. In selecting a cement for sulfate resistance, the principal consideration is its C_3A content. For moderate exposures, Type II cement is limited to a maximum C_3A content of 8.0 percent under ASTM C 150. The blended cements under ASTM C 595 made with portland cement clinker with less than 8 percent C_3A qualify for the MS designation, and therefore, are appropriate for use in moderate sulfate exposures. The appropriate types under ASTM C 595 are IP(MS), IS(MS), IP(M)(MS), and IS(M)(MS). For severe exposures, Type V cement with a maximum C_3A content of 5 percent is specified. In certain areas, the C_3A content of other available types such as Type III or Type I may be less than 8 or 5 percent and are usable in moderate or severe sulfate exposures. Note that sulfate-resisting cement will not increase resistance to some chemically aggressive solutions, for example ammonium nitrate. The project specifications should cover all special cases.

The judicious employment of a good quality fly ash (ASTM C 618, Class F) also has been shown to improve the sulfate resistance of concrete.[4.2] According to recent research, certain Type IP cements made with Class F pozzolan and portland cement with a tricalcium aluminate (C_3A) content greater than 8 percent can provide sulfate resistance for moderate exposures.

A note to Table 4.2.1 lists seawater as "moderate exposure," even though it generally contains more than 1500 ppm SO_4. Recent data further indicate that for seawater

TABLE 4.2.1 — REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Normal weight aggregate concrete Maximum water-cement ratio, by weight*	Lightweight aggregate concrete Minimum compressive strength, f'_c psi*
Negligible	0.00–0.10	0–150	—	—	—
Moderate†	0.10–0.20	150–1500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	3750
Severe	0.20–2.00	1500–10,000	V	0.45	4250
Very severe	Over 2.00	Over 10,000	V plus pozzolan‡	0.45	4250

*A lower water-cement ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 4.1.2).

†Seawater.

‡Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

CODE

4.2.2 – Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 4.2.1.

4.3 – Corrosion of reinforcement

4.3.1 – For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed the limits of Table 4.3.1. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to AASHTO T 260.

TABLE 4.3.1 — MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION

Type of member	Maximum water soluble chloride ion (Cl^-) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

COMMENTARY

exposures, other types of cement with C_3A up to 10 percent may be used if the maximum water-cement ratio is further reduced to 0.40.

In addition to the proper selection of cement, other requirements for durable concrete exposed to concentrations of sulfate such as: low water-cement ratio (minimum strength for lightweight aggregate concrete), adequate air entrainment, low slump, adequate consolidation, uniformity, adequate cover of reinforcement, and sufficient moist curing to develop the potential properties of the concrete, are essential.

R4.3 – Corrosion of reinforcement

Additional information on the effects of chlorides on the corrosion of reinforcing steel is given in **"Guide to Durable Concrete"** reported by ACI Committee 201[4.3] and **"Corrosion of Metals in Concrete"** reported by ACI Committee 222.[4.4] Test procedures must conform to those given in AASHTO Method T 260.[4.5] An initial evaluation may be obtained by testing individual concrete ingredients for total chloride ion content. If total chloride ion content, calculated on the basis of concrete proportions, exceeds those permitted in Table 4.3.1, it may be necessary to test samples of the hardened concrete for water soluble chloride ion content described in the guide. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described.

When concretes are tested for soluble chloride ion content the tests should be made at an age of 28 to 42 days. The limits in Table 4.3.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

The chloride ion limits in Table 4.3.1 differ from those recommended in ACI 201 and ACI 222. For reinforced concrete that will be dry in service, a limit of one percent has been included to control total soluble chlorides. Table 4.3.1 includes limits of 0.15 and 0.30 percent for reinforced concrete that will be exposed to chlorides or will be damp in service, respectively. These limits compare to 0.10 and 0.15 recommended in ACI 201. ACI 222 recommends limits of 0.08 and 0.20 percent by weight of cement for chlorides in prestressed and reinforced concrete, respectively, based on tests for acid soluble chlorides, not the test for water soluble chlorides required here.

When epoxy or zinc-coated bars are used, the limits in Table 4.3.1 may be more restrictive than necessary.

CODE

COMMENTARY

4.3.2 – When reinforced concrete will be exposed to deicing salts, brackish water, seawater, or spray from these sources, requirements of Table 4.1.2 for water-cement ratio or concrete strength and minimum concrete cover requirements of 7.7 shall be satisfied. See 18.14 for unbonded prestressing tendons.

R4.3.2 – When concretes are exposed to external sources of chlorides, the cover required in 7.7 may need to be increased or epoxy-coated reinforcement provided. The designer should evaluate conditions in parking structures where salt may be tracked in by vehicles or in structures on or near the ocean.

7.5 – Placing reinforcement

7.5.1 – Reinforcement, prestressing tendons, and ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 7.5.2.

7.5.2 – Unless otherwise specified by the Engineer, reinforcement, prestressing tendons, and prestressing ducts shall be placed within the following tolerances:

7.5.2.1 – Tolerance for depth ***d***, and minimum concrete cover in flexural members, walls and compression members shall be as follows:

	Tolerance on ***d***	Tolerance on minimum concrete cover
$d \leq 8$ in.	±3/8 in.	−3/8 in.
$d > 8$ in.	±1/2 in.	−1/2 in.

Except that tolerance for the clear distance to formed soffits shall be minus 1/4 in. and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover required in the design drawings or specifications.

R7.5 – Placing reinforcement

R7.5.2 – Generally accepted practice, as reflected in other ACI standards, has established tolerances on total depth (formwork or finish) and fabrication of truss bent reinforcing bars and closed ties, stirrups, and spirals. The Engineer should specify closer tolerances than those permitted by the code which may be necessary to minimize the accumulation of tolerances resulting in excessive reduction in effective depth or cover.

A tighter tolerance has been placed on minimum clear distance to formed soffits because of its importance for durability and fire protection, and because bars are usually supported in such a manner that the specified tolerance is practical.

Closer tolerances than those required by the code may be desirable for prestressed concrete to achieve camber control within limits acceptable to the designer or owner. In such cases, the engineer should specify the necessary tolerances. Recommendations are given in Reference 7.5.

7.7 – Concrete protection for reinforcement

7.7.1 – Cast-in-place concrete (nonprestressed)

The following minimum concrete cover shall be provided for reinforcement:

	Minimum cover, in.
(a) Concrete cast against and permanently exposed to earth...	3
(b) Concrete exposed to earth or weather: [Note: See code interpretation page ii.]	
#6 through #18 bars........	2
#5 bar, W31 or D31 wire, and smaller..................	1-1/2
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
#14 and #18 bars.........	1-1/2
#11 bar and smaller	3/4
Beams, columns:	
Primary reinforcement, ties, stirrups, spirals...........	1-1/2
Shells, folded plate members:	
#6 bar and larger.........	3/4
#5 bar, W31 or D31 wire, and smaller	1/2

R7.7 – Concrete protection for reinforcement

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where minimum cover is prescribed for a class of structural member, it is measured to the outer edge of stirrups, ties, or spirals if transverse reinforcement encloses main bars; to the outermost layer of bars if more than one layer is used without stirrups or ties; to the metal end fitting or duct on post-tensioned prestressing steel.

The condition "concrete surfaces exposed to the weather" refers to direct exposure to moisture changes and not just to temperature changes. Slab or thin shell soffits are not usually considered directly "exposed" unless subject to alternate wetting and drying, including that due to condensation conditions or direct leakage from exposed top surface, run off, or similar effects.

The development lengths given in Chapter 12 are now a function of the bar cover. As a result, it may be desirable to use larger than minimum cover in some cases.

CODE

7.7.2 – Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for reinforcement:

(a) Concrete exposed to earth or weather:	
Wall panels:	
#14 and #18 bars	1-1/2
#11 bar and smaller	3/4
Other members:	
#14 and #18 bars	2
#6 through #11 bars	1-1/2
#5 bar, W31 or D31 wire, and smaller	1-1/4
(b) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
#14 and #18 bars	1-1/4
#11 bar and smaller	5/8
Beams, columns:	
Primary reinforcement	d_b but not less than 5/8 and need not exceed 1-1/2
Ties, stirrups, spirals	3/8
Shells, folded plate members:	
#6 bar and larger.	5/8
#5 bar, W31 or D31 wire, and smaller	3/8

7.7.3 – Prestressed concrete

7.7.3.1 – The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, except as provided in 7.7.3.2 and 7.7.3.3:

	Minimum cover, in.
(a) Concrete cast against and permanently exposed to earth. . .	3
(b) Concrete exposed to earth or weather:	
Wall panels, slabs, joists	1
Other members	1-1/2
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists	3/4.
Beams, columns:	
Primary reinforcement	1-1/2
Ties, stirrups, spirals	1

COMMENTARY

R7.7.2 – The lesser thicknesses for precast construction reflect the greater convenience of control for proportioning, placing, and curing inherent in precasting. The term "manufactured under plant controlled conditions" does not specifically imply that precast members must be manufactured in a plant. Structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedure are equal to that normally expected in a plant.

CODE

Shells, folded plate members:
#5 bar, W31 or D31 wire,
and smaller 3/8
Other reinforcement.........d_b but not less than 3/4

7.7.3.2 – For prestressed concrete members exposed to earth, weather, or corrosive environments, and in which permissible tensile stress of 18.4.2(b) is exceeded, minimum cover shall be increased 50 percent.

7.7.3.3 – For prestressed concrete members manufactured under plant control conditions, minimum concrete cover for nonprestressed reinforcement shall be as required in 7.7.2.

7.7.4 – Bundled bars

For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 in.; except for concrete cast against and permanently exposed to earth, minimum cover shall be 3 in.

7.7.5 – Corrosive environments

In corrosive environments or other severe exposure conditions, amount of concrete protection shall be suitably increased, and denseness and nonporosity of protecting concrete shall be considered, or other protection shall be provided.

e.g. epoxy coated bars

COMMENTARY

R7.7.5 – Corrosive environments

When concrete will be exposed to external sources of chlorides in service, such as deicing salts, brackish water, seawater, or spray from these sources, concrete must be proportioned to satisfy the special exposure requirements of code Chapter 4. These include minimum air content, maximum water-cement ratio (or minimum strength for lightweight concrete), maximum chloride ion in concrete, and cement type. Additionally, for corrosion protection, a minimum concrete cover for reinforcement of 2 in. for walls and slabs and 2-1/2 in. for other members is recommended. For precast concrete manufactured under plant control conditions, a minimum cover of 1-1/2 and 2 in., respectively, is recommended.

STANDARD SPECIFICATIONS for HIGHWAY BRIDGES

FIFTEENTH EDITION

1992

STEEL

Part B
DESIGN DETAILS

10.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

10.3.1 Allowable Fatigue Stress

Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Article 10.32 and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 10.3.1A for the appropriate type and location of material given in Table 10.3.1B and shown in Figure 10.3.1C.

For unpainted weathering steel, A709, all grades, the values of allowable fatigue stress range, Table 10.3.1A, as modified by footnote d, are valid only when the design and details are in accordance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.

Main load carrying components subjected to tensile stresses that may be considered nonredundant load path members—that is, where failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 10.3.1A for Nonredundant Load Path Structures. Examples of nonredundant load path members are flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two-column bents.

TABLE 10.3.1A Allowable Fatigue Stress Range

Redundant Load Path Structures*

Category (See Table 10.3.1B)	Allowable Range of Stress, F_{sr} (ksi)[a] For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	63 (49)[d]	37 (29)[d]	24 (18)[d]	24 (16)[d]
B	49	29	18	16
B′	39	23	14.5	12
C	35.5	21	13	10
				12[b]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8

Nonredundant Load Path Structures

Category (See Table 10.3.1B)	Allowable Range of Stress, F_{sr} (ksi)[a] For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	50 (39)[d]	29 (23)[d]	24 (16)[d]	24 (16)[d]
B	39	23	16	16
B′	31	18	11	11
C	28	16	10	9
			12[b]	11[b]
D	22	13	8	5
E[c]	17	10	6	2.3
E′	12	7	4	1.3
F	12	9	7	6

* Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

[a] The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

[b] For transverse stiffener welds on girder webs or flanges.

[c] Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures.

[d] For unpainted weathering steel, A709, all grades, when used in conformance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.

10.3.2 Load Cycles

10.3.2.1 The number of cycles of maximum stress range to be considered in the design shall be selected from Table 10.3.2A unless traffic and loadometer surveys or other considerations indicate otherwise.

10.3.2.2 Allowable fatigue stresses shall apply to those Group Loadings that include live load or wind load.

10.3.2.3 The number of cycles of stress range to be considered for wind loads in combination with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

10.3.3 Charpy V-Notch Impact Requirements

10.3.3.1 Main load carrying member components subjected to tensile stress require supplemental impact properties as described in the Material Specifications.**

10.3.3.2 These impact requirements vary depending on the type of steel, type of construction, welded or mechanically fastened, and the average minimum service temperature to which the structure may be subjected.*** Table 10.3.3A contains the temperature zone designations.

10.3.3.3 Components requiring mandatory impact properties shall be designated on the drawings and the appropriate zone shall be designated in the contract documents.

10.3.3.4 M 270 Grades 100/100W steel shall be supplied to Zone 2 requirements as a minimum.

10.3.4 Shear

10.3.4.1 When longitudinal beam or girder members in bridges designed for Case I roadways are investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge (see footnote c of Table 10.3.2A), the total shear force in the beam or girder under this single-truck loading shall be limited to $0.58\ F_yDt_wC$. The constant C, the ratio of the buckling shear stress to the shear yield stress is defined in Article 10.34.4.2 or Article 10.48.8.1.

** AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*.

*** The basis and philosophy used to develop these requirements are given in a paper entitled "The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels" by John M. Barsom, February 1975, available from the American Iron and Steel Institute, Washington, D.C.

TABLE 10.3.1B

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
Plain Member	Base metal with rolled or cleaned surface. Flame cut edges with ANSI smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds (with backing bars removed) or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3,4,5,7
	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds with backing bars not removed, or by continuous partial penetration groove welds parallel to the direction of applied stress.	T or Rev	B'	3,4,5,7
	Calculated flexural stress at the toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends, or wider than flange with welds across the ends:			
	(a) Flange thickness ≤ 0.8 in.	T or Rev	E	7
	(b) Flange thickness > 0.8 in.	T or Rev	E'	7
	Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	T or Rev	E'	7
Groove Welded Connections	Base metal and weld metal in or adjacent to full penetration groove weld splices of rolled or welded sections having similar profiles when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	8,10
	Base metal and weld metal in or adjacent to full penetration groove weld splices with 2 ft. radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	13
	Base metal and weld metal in or adjacent to full penetration groove weld splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of the applied stress, and weld soundness established by nondestructive inspection:			
	(a) AASHTO M270 Grades 100/100W (ASTM A709) base metal	T or Rev	B'	11,12
	(b) Other base metals	T or Rev	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove weld splices, with or without transitions having slopes no greater than 1 to 2½, when the reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8,10,11,12
Groove Welded Attachments—Longitudinally Loaded[b]	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 in.	T or Rev	C	6,15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15

TABLE 10.3.1B (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	15
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	15
	Base metal adjacent to details attached by full or partial penetration groove welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without end welds ground smooth.	T or Rev	E	16
Groove welded Attachments—Transversely Loaded[b,c]	Detail base metal attached by full penetration groove welds with a transition radius, R, regardless of the detail length and with weld soundness transverse to the direction of stress established by nondestructive inspection:			
	—With equal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—With equal plate thickness and reinforcement not removed	T or Rev		16
	(a) Transition radius ≥ 6 in.		C	
	(b) 6 in. > Transition radius ≥ 2 in.		D	
	(c) 2 in. > Transition radius ≥ 0 in.		E	
	—With unequal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii with unequal plate thickness and reinforcement not removed.	T or Rev	E	16
Fillet Welded Connections	Base metal at details connected with transversely loaded welds, with the welds perpendicular to the direction of stress:			
	(a) Detail thickness ≤ 0.5 in.	T or Rev	C	14
	(b) Detail thickness > 0.5 in.	T or Rev	See Note[d]	
	Base metal at intermittent fillet welds.	T or Rev	E	—
	Shear stress on throat of fillet welds.	Shear	F	9
Fillet Welded Attachments—Longitudinally Loaded[b,c,e]	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, is less than 2 in. and stud-type shear connectors.	T or Rev	C	15,17,18,20
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15,17
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	7,9,15,17
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	7,9,15

TABLE 10.3.1B (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Fillet Welded Attachments—Transversely Loaded with the weld in the direction of principal stress[b,e]	Detail base metal attached by fillet welds with a transition radius, R, regardless of the detail length (shear stress on the throat of fillet welds governed by Category F):			
	—With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Mechanically Fastened Connections	Base metal at gross section of high strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connecting materials.	T or Rev	B	21
	Base metal at net section of high strength bolted bearing-type connections.	T or Rev	B	21
	Base metal at net section of riveted connections.	T or Rev	D	21

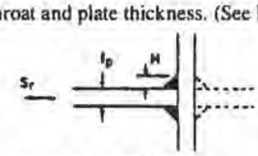
[a] "T" signifies range in tensile stress only, "Rev" signifies a range of stress involving both tension and compression during a stress cycle.

[b] "Longitudinally Loaded" signifies direction of applied stress is parallel to the longitudinal axis of the weld. "Transversely Loaded" signifies direction of applied stress is perpendicular to the longitudinal axis of the weld.

[c] Transversely loaded partial penetration groove welds are prohibited.

[d] Allowable fatigue stress range on throat of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, Journal of the Structural Division, ASCE, Vol. 105, No. ST9, Sept. 1979.)

$$S_r = S_r^C \left(\frac{0.06 + 0.79H/t_p}{1.1 t_p^{1/6}} \right)$$



where S_r^C is equal to the allowable stress range for Category C given in Table 10.3.1A. This assumes no penetration at the weld root.

[e] Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.

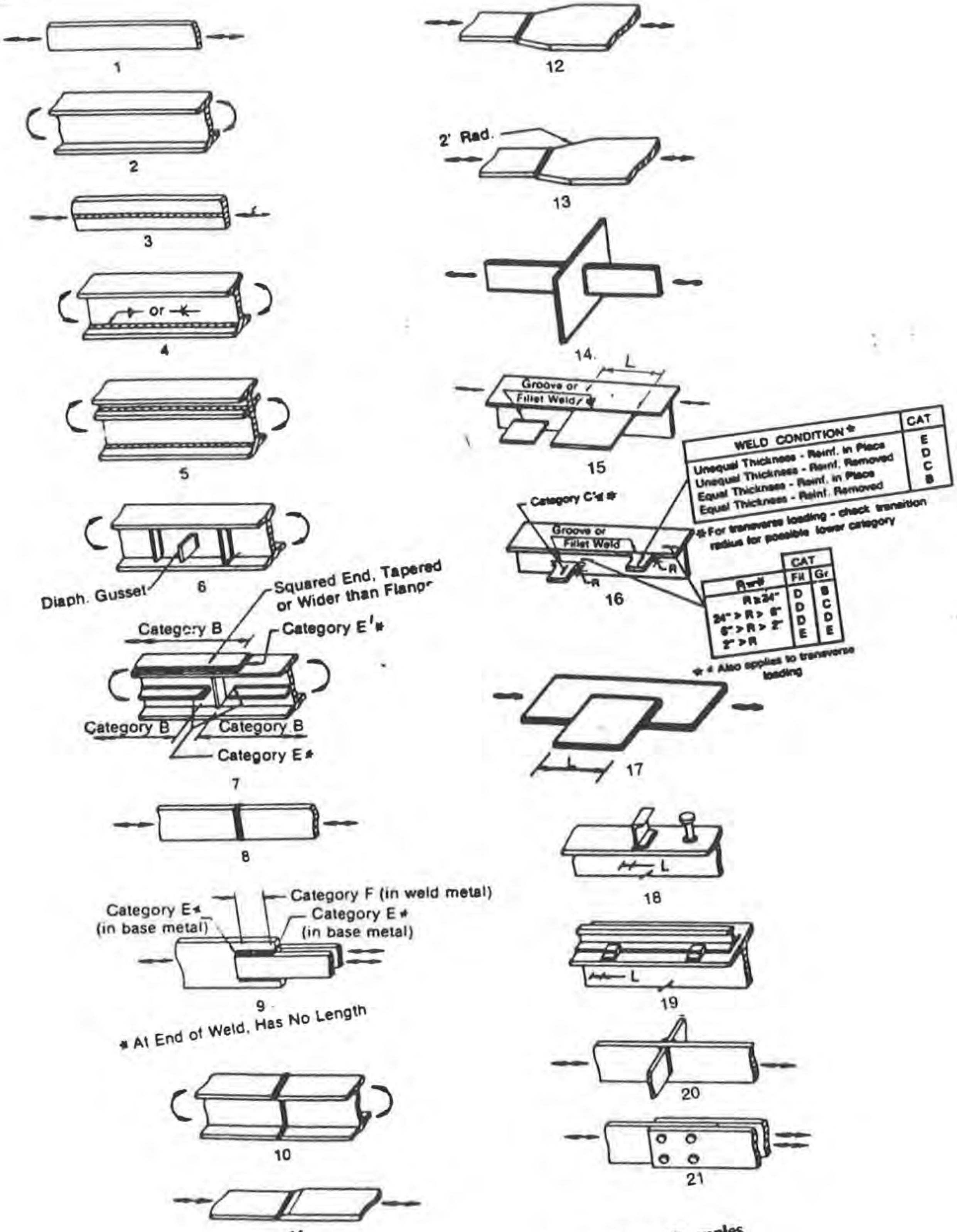


Figure 10.3.1C. Illustrative Examples

TABLE 10.3.2A Stress Cycles

Main (Longitudinal) Load Carrying Members				
Type of Road	Case	ADTT[a]	Truck Loading	Lane Loading[b]
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	2,000,000[c]	500,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000

Transverse Members and Details Subjected to Wheel Loads			
Type of Road	Case	ADTT[a]	Truck Loading
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	over 2,000,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	2,000,000
Other Highways and Streets	III	—	500,000

[a] Average Daily Truck Traffic (one direction).
[b] Longitudinal members should also be checked for truck loading.
[c] Members shall also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading. The shear in steel girder webs shall not exceed 0.58 F_yDt_wC for this single truck loading.

TABLE 10.3.3A Temperature Zone Designations for Charpy V-Notch Impact Requirements

Minimum Service Temperature	Temperature Zone Designation
0°F and above	1
−1°F to −30°F	2
−31°F to −60°F	3

10.32.3.4 Fatigue

When subject to tensile fatigue loading, the tensile stress in the bolt due to the service load plus the prying force resulting from application of service load shall not exceed the following design stresses in kips per square inch. The nominal diameter of the bolt shall be used in calculating the bolt stress. The prying force shall not exceed 80 percent of the externally applied load.

Number of Cycles	AASHTO M164 (ASTM A325)	AASHTO M253 (ASTM A490)
Not more than 20,000	39.5	48.5
From 20,000 to 500,000	35.5	44.0
More than 500,000	27.5	34.0

13.2 PAINTING METAL STRUCTURES

13.2.1 Coating Systems and Paints

The coating system and paints to be applied shall consist of the system in Table 13.2.1 which is specified for use or modified by the special provisions.

TABLE 13.2.1

	High Pollution and Coastal	Mild Climate	Mild Climate or Maint. Repainting
Primer	Inorganic Zinc—3 mils.	Organic Zinc—3 mils.	Oil/Alkyd—2 mils.
Intermediate Coat	Epoxy 2 mils or Vinyl Wash Primer 0.3-0.5 mils.	Epoxy 2 mils or Vinyl Wash Primer 0.3-0.5 mils.	Oil/Alkyd—2 mils.
Top Coat	Epoxy, Vinyl, or Urethane—2 mils.	Epoxy, Vinyl, or Urethane—2 mils.	Oil/Alkyd—2 mils.
Total System	5.3-7 mils.	5.3-7 mils.	6 mils.

Notes:

(1) Except for vinyl wash primer, the coating and system thicknesses shown are minimums.

(2) Coating systems shown for severe areas are satisfactory in less severe areas.

(3) Coastal—within 1,000 feet of ocean or tidal water. High Pollution—air pollution environment such as industrial areas. Mild—other than coastal area not in air pollution environment.

(4) Inorganic zinc paint shall meet the requirements of Military Specification DOD-P-23236A (SH).

(5) Organic zinc paint shall meet the requirements of Military Specification DOD-P-21035A.

(6) Vinyl Wash Primer shall meet the requirements of Military Specification DOD-P-15328D.

(7) Vinyl top coat paint shall meet the requirements of the Steel Structures Painting Council, SSPC-Paint 9.

(8) Epoxy paint shall meet the requirements of the Steel Structures Painting Council, SSPC-22.

(9) Oil/Alkyd primer and intermediate coat paint shall meet the requirements of the Steel Structures Painting Council, SSPC-Paint 25.

(10) Oil/Alkyd top coat paint shall meet the requirements of the Steel Structures Painting Council, SSPC-Paint 104.

(11) Urethane top coat paint shall meet the recommendations of the Steel Structures Painting Council, SSPC-PS Guide 1700.

(12) Paints are hazardous because of their flammability and potential toxicity. Safe handling practices are required and should include, but not be limited to, the provisions of SSPC-PA Guide 3, "A Guide to Safety in Paint Application."

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K4. FATIGUE

Members and their connections subject to fatigue loading shall be proportioned in accordance with the provisions of Appendix K4.

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

CHAPTER L

SERVICEABILITY DESIGN CONSIDERATIONS

This chapter provides design guidance for serviceability considerations not covered elsewhere. Serviceability is a state in which the function of a building, its appearance, maintainability, durability and comfort of its occupants are preserved under normal usage.

Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure.

L1. CAMBER

If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, the requirements shall be set forth in the design documents.

Trusses of 80 ft or greater span generally shall be cambered for approximately the dead-load deflection. Crane girders of 75 ft or greater span generally shall be cambered for approximately the dead-load deflection plus ½ the live-load deflection.

Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward. If camber involves the erection of any member under a preload, this shall be noted in the design documents.

L2. EXPANSION AND CONTRACTION

Provision shall be made for expansion and contraction appropriate to the service conditions of the structure.

L3. DEFLECTION, VIBRATION AND DRIFT

1. Deflection

Beams and girders supporting floors and roofs shall be proportioned with due regard to the deflection produced by the design loads. Beams and girders supporting plastered ceilings shall be so proportioned that the maximum live-load deflection does not exceed $\frac{1}{360}$ of the span.

2. Vibration

Beams and girders supporting large open floor areas free of partitions or other sources of damping shall be designed with due regard for vibration.

L4. CONNECTION SLIP

For the design of slip-resistant connections see Sect. J3.

L5. CORROSION

When appropriate, structural components shall be designed to tolerate corrosion or shall be protected against corrosion that impairs the strength or serviceability of the structure.

Where beams are exposed they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and painting.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the *Code of Standard Practice* of the American Institute of Steel Construction, Inc.

Unless otherwise specified, steelwork which will be concealed by interior building finish or will be in contact with concrete need not be painted. Unless specifically excluded, all other steelwork shall be given one coat of shop paint.

2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

3. Contact Surfaces

Paint is permitted unconditionally in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, paragraph 3.(b).

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust-inhibiting coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. of any field weld location shall be free of materials that would prevent proper welding or produce toxic fumes during welding.

6. Field Painting

Responsibility for touch-up painting, cleaning and field-painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

APPENDIX K

STRENGTH DESIGN CONSIDERATIONS

K4. FATIGUE

Members and connections subject to fatigue loading shall be proportioned in accordance with the provisions of this Appendix.

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangement of live load.

1. Loading Conditions; Type and Location of Material

In the design of members and connections subject to repeated variation of live load, consideration shall be given to the number of stress cycles, the expected range of stress and the type and location of member or detail.

Loading conditions shall be classified according to Table A-K4.1.

The type and location of material shall be categorized according to Table A-K4.2.

2. Allowable Stress Range

The maximum stress shall not exceed the basic allowable stress provided in Chapters A through M of this Specification and the maximum range of stress shall not exceed that given in Table A-K4.1.

TABLE A-K4.1
Number of Loading Cycles

Loading Condition	From	To
1	20,000[a]	100,000[b]
2	100,000	500,000[c]
3	500,000	2,000,000[d]
4	Over 2,000,000	

[a]Approximately equivalent to two applications every day for 25 years.
[b]Approximately equivalent to 10 applications every day for 25 years.
[c]Approximately equivalent to 50 applications every day for 25 years.
[d]Approximately equivalent to 200 applications every day for 25 years

3. Tensile Fatigue

When subject to tensile fatigue loading, the tensile stress in A325 or A490 bolts due to the combined applied load and prying forces shall not exceed the following values, and the prying force shall not exceed 60% of the externally applied load.

Number of Cycles	A325	A490
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

Bolts must be tensioned to the requirements of Table J3.7.

The use of other bolts and threaded parts subjected to tensile fatigue loading is not recommended.

TABLE A-K4.2
Stress Category Classifications

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Plain Material	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less	T or Rev.	A	1,2
Built-up Members	Base metal in members without attachments, built-up plates or shapes connected by continuous full-penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,6
	Base metal in members without attachments, built-up plates, or shapes connected by full-penetration groove welds with backing bars not removed, or by partial-penetration groove welds parallel to the direction of applied stress	T or Rev.	B′	3,4,5,6
	Base metal at toe welds on girder webs or flanges adjacent to welded transverse stiffeners	T or Rev.	C	7
	Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than flange with welds across the ends			
	Flange thickness $\leq$ 0.8 in.	T or Rev.	E	5
	Flange thickness $>$ 0.8 in.	T or Rev.	E′	5
	Base metal at end of partial length welded cover plates wider than the flange without welds across the ends		E′	5

[a] "T" signifies range in tensile stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear, including shear stress reversal.
[b] These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.
[c] Allowable fatigue stress range for transverse partial-penetration and transverse fillet welds is a function of the effective throat, depth of penetration and plate thickness. See Frank and Fisher (1979).

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illus-trative Example Nos. (see Fig. A-K4.1)[b]
Groove Welds	Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1-85	T or Rev.	B	10,11
	Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1-85			
	A514 base metal	T or Rev.	B'	12,13
	Other base metals	T or Rev.	B	12,13
	Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed but weld soundness is established by radiographic or ultrasonic inspection in accordance with requirements of 9.25.2 or 9.25.3 of AWS D1.1-85	T or Rev.	C	10,11,12, 13
Partial-Penetration Groove Welds	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds	T or Rev.	F[c]	16

TABLE A-K4.2 (cont'd) Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Fillet-welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	
	Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses $b \leq 1$ in. $b > 1$ in.	 T or Rev. T or Rev.	 E E'	 17,18 17,18
	Base metal at members connected with transverse fillet welds $b \leq \frac{1}{2}$ in. $b > \frac{1}{2}$ in.	 T or Rev.	 C See Note c	 20,21
Fillet Welds	Weld metal of continuous or intermittent longitudinal or transverse fillet welds	S	F[c]	15,17,18 20,21
Plug or Slot Welds	Base metal at plug or slot welds	T or Rev.	E	27
	Shear on plug or slot welds	S	F	27
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip-critical connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	8
	Base metal at net section of other mechanically fastened joints	T or Rev.	D	8,9
	Base metal at net section of fully tensioned high-strength, bolted-bearing connections	T or Rev.	B	8,9

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments	Base metal at details attached by full-penetration groove welds subject to longitudinal and/or transverse loading when the detail embodies a transition radius *R* with the weld termination ground smooth and for transverse loading, the weld soundness established by radiographic or ultrasonic inspection in accordance with 9.25.2 or 9.25.3 of AWS D1.1-85			
	Longitudinal loading			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R > 6$ in.	T or Rev.	C	14
	6 in. $> R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14
	Detail base metal for transverse loading: equal thickness and reinforcement removed			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R > 6$ in.	T or Rev.	C	14
	6 in. $> R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: equal thickness and reinforcement not removed			
	$R > 24$ in.	T or Rev.	C	14
	24 in. $> R > 6$ in.	T or Rev.	C	14
	6 in. $> R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illus-trative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Detail base metal for transverse loading: unequal thickness and reinforcement removed			
	$R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: unequal thickness and reinforcement not removed			
	all R	T or Rev.	E	14,15
	Detail base metal for transverse loading			
	$R > 6$ in.	T or Rev.	C	19
	6 in. $> R >$ 2 in.	T or Rev.	D	19
	2 in. $> R$	T or Rev.	E	19
	Base metal at detail attached by full-penetration groove welds subject to longitudinal loading			
	$2 < a < 12b$ or 4 in.	T or Rev.	D	15
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E'	15
	Base metal at detail attached by fillet welds or partial-penetration groove welds subject to longitudinal loading			
	$a < 2$ in.	T or Rev.	C	15,23,24, 25,26
	2 in. $< a < 12b$ or 4 in.	T or Rev.	D	15,23, 24,26
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15,23, 24,26
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E'	15,23, 24,26

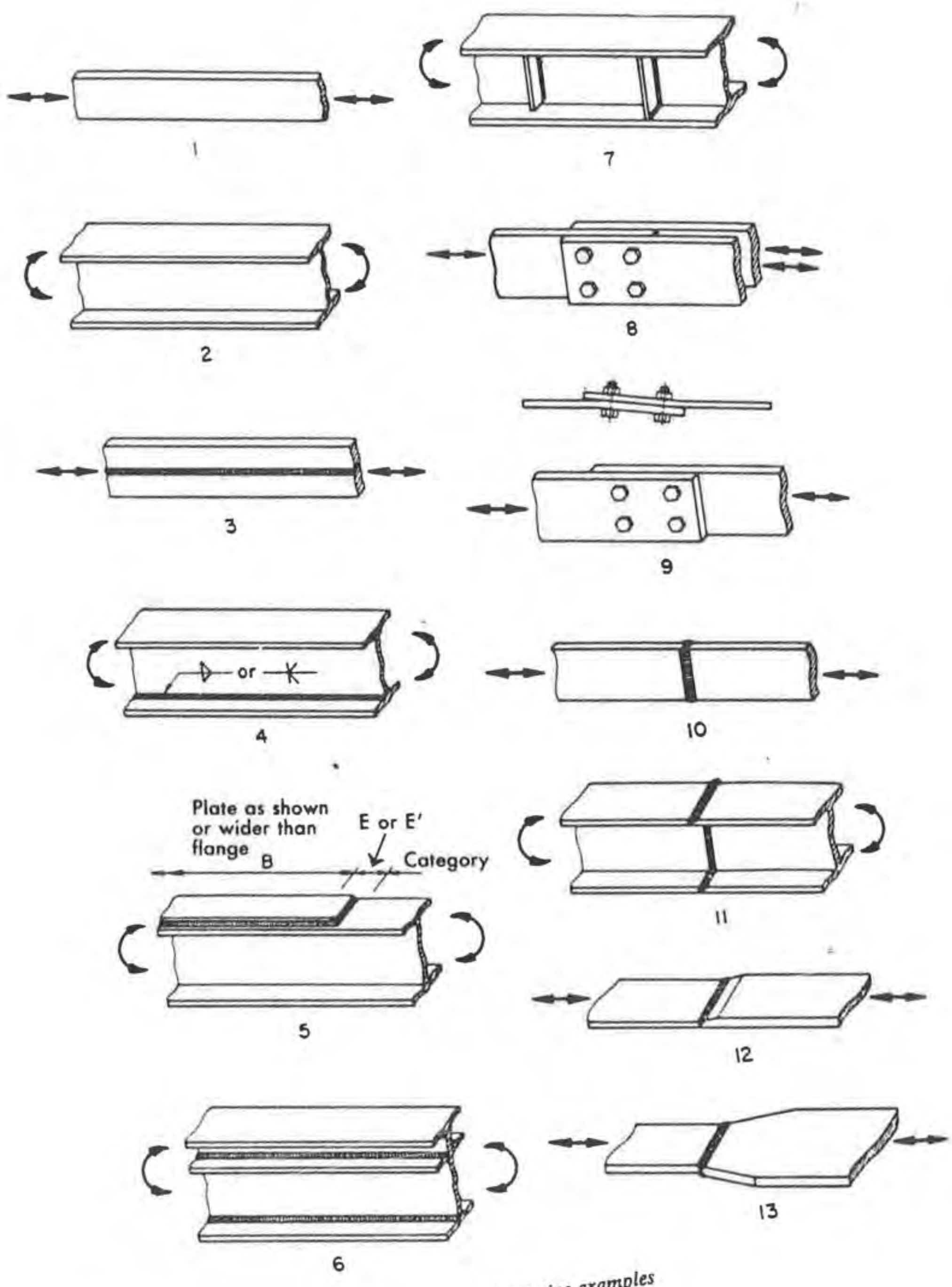
TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Base metal attached by fillet welds or partial-penetration groove welds subjected to longitudinal loading when the weld termination embodies a transition radius with the weld termination ground smooth:			
	$R > 2$ in.	T or Rev.	D	19
	$R \leq 2$ in.	T or Rev	E	19
	Fillet-welded attachments where the weld termination embodies a transition radius, weld termination ground smooth, and main material subject to longitudinal loading:			
	Detail base metal for transverse loading:			
	$R > 2$ in.	T or Rev.	D	19
	$R < 2$ in.	T or Rev.	E	19
	Base metal at stud-type shear connector attached by fillet weld or automatic end weld	T or Rev.	C	22
	Shear stress on nominal area of stud-type shear connectors	S	F	

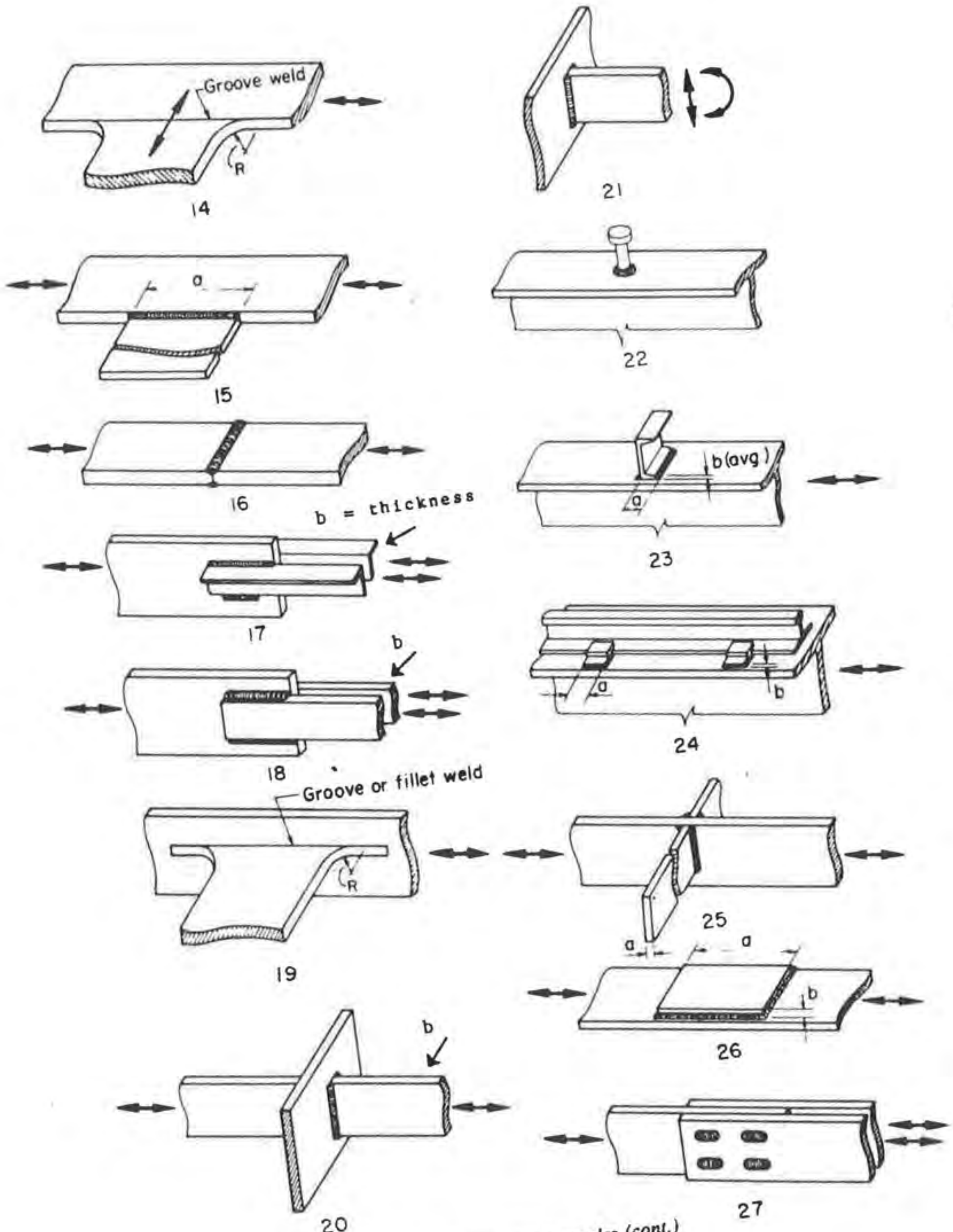
TABLE A-K4.3
Allowable Stress Range, Ksi

Category (from Table A-K4.2)	Loading Condition 1	Loading Condition 2	Loading Condition 3	Loading Condition 4
A	63	37	24	24
B	49	29	18	16
B'	39	23	15	12
C	35	21	13	10[a]
D	28	16	10	7
E	22	13	8	5
E'	16	9	6	3
F	15	12	9	8

[a] Flexural stress range of 12 ksi permitted at toe of stiffener welds on flanges



Illustrative examples



Illustrative examples (cont.)

Appendix B

Bridge Durability/Longevity
Transmittal Letter and Questionnaire

Auburn University

Auburn University, Alabama 36849-5337

College of Engineering

Department of Civil Engineering
Harbert Engineering Center

Telephone: (205) 844-4320
Fax: (205) 844-6290

December 1, 1992

Dear __________:

I am writing to solicit your help on a research project that I feel is important to all of us, "Bridge Durability/Longevity." Because of the large number of bridges that currently need major rehabilitation or replacement, the large dollar investment that this represents, and the fact that durability of our current highway bridges leaves something to be desired, it behooves us to take a close look at what we have been and/or are currently doing in the areas of bridge planning, design, construction and maintenance regarding bridge durability. This will help identify needed changes in each area in order to produce bridges in the future with greater durability/longevity.

The Highway Research Center at Auburn University recognizes the validity and importance of the above statements and has initiated a "state-of-the-art"/historical record review type of investigation to address this issue. The objective of this investigation is "to discover the nature, extent and causes of highway bridge deteriorations in Alabama, and to assess their relative frequency, scope and importance." In addressing this objective, we recognize that the existing literature has much to offer on the topic and are of course searching this source. We also recognize that state and county engineers, and particularly maintenance engineers in these agencies, throughout the state represent a huge source of knowledge on this topic. In an effort to tap this resource, we have prepared the enclosed Bridge Durability and Longevity Survey Questionnaire.

I know that your people have many demands on their time. However, because of the importance of this topic and the need to begin addressing it immediately, and because of the unique experience, expertise and guidance that your Division Maintenance Engineer and Chief Bridge Inspector can offer, I am requesting that you ask each of these people to complete one of the enclosed questionnaires. Their input will be most helpful to us. Also, if there are other people in your Division whom you feel can offer good expertise and guidance on this topic, will you please make additional copies of the questionnaire and ask them to respond as well. We plan to start analyzing and synthesizing the responses in December 1992 and hence request that you have them return the questionnaire by December 18, 1992.

Thank you in advance for your help in this important matter. Please call me at the number on this letterhead if you have any questions about the questionnaire or if you would like to offer further guidance on this subject.

Sincerely,

G.E. Ramey
Feagin Professor
of Civil Engineering

2-Enclosures

Survey Questionnaire
to Identify Areas of Focus to Achieve Greater
Bridge Durability and Longevity

Instructions

It is understood that good planning, design, construction and maintenance are all vitally important for a bridge to give excellent service and longevity. What is being asked in the questions below is - "Is additional attention needed (above and beyond current 1992 practices) in various areas to achieve greater bridge durability/longevity?" We ask that you keep this in mind as you answer each question.

Questions

1. I believe that additional attention to bridge longevity in the bridge planning phase (i.e., projection of needed traffic capacity, number of lanes, need for shoulder, flood water levels, etc.) is currently needed to achieve significantly greater bridge durability/longevity.

   ☐ Yes
   ☐ No

   Please check the areas below where you think additional attention would pay good dividends in bridge longevity.

   ☐ Planning traffic volume and the number of lanes required
   ☐ Planning lane and shoulder widths
   ☐ Planning horizontal or vertical alignment
   ☐ Planning foundations and clearances for flood frequency and flood water levels
   ☐ Planning for streambed stability (scour)
   ☐ Other (please specify) ____________________
   ____

2. I believe that additional attention to bridge durability in the detailed structural design phase (selection of bridge material, design loads, bridge type, spans, continuous vs. simple span, geometry and member sizing, substructure, foundations, etc.) is currently needed to achieve significantly greater bridge durability/longevity.

   ☐ Yes
   ☐ No

   Please check the areas below where you think additional attention would pay good dividends in bridge durability.

   ☐ Projection and selection of more appropriate design live loads, i.e., traffic loads (both volume of traffic and loading levels)

- ☐ Selection and specification of bridge material type, strength, and other material properties
- ☐ Selection of bridge type/structural form
- ☐ Selection of continuous vs. simple span
- ☐ Selection of straight vs. skewed or horizontally curved girders
- ☐ Engineering of bridge, member and connection/joint details to reduce stress concentrations and fatigue damage at fatigue sensitive locations
- ☐ Engineering of bridge joints
- ☐ Engineering of bridge support bearings
- ☐ Engineering of bridge for enhanced constructability
- ☐ Engineering of bridge for enhanced future maintainability and inspectability
- ☐ Superstructure design for corrosion resistance
- ☐ Superstructure design for impact damage from highway traffic
- ☐ Bridge deck design for enhanced durability
- ☐ Bridge diaphragm to girder connections, transverse X-bracing, etc.
- ☐ Substructure structural design
- ☐ Substructure design for river meandering
- ☐ Substructure design for scour at bridge foundations
- ☐ Substructure design for corrosion resistance
- ☐ Substructure design for impact damage from river traffic
- ☐ Substructure design to resist buildup of debris
- ☐ Other (please specify) ____________________________
  ____

3. **I believe that additional attention to bridge durability in the <u>construction phase</u> is currently needed to achieve significantly greater bridge durability/longevity.**

- ☐ Yes
- ☐ No

Please check the areas below where you think <u>additional attention</u> would pay good dividends in bridge durability.

- ☐ Achieving the specified bridge material strengths, densities, and other material properties in-place in the constructed bridge.

- ☐ Construction quality to have the "as built" bridge match the "as designed" bridge regarding rebar locations, member sizes, bridge joint/connection details, etc.

- ☐ On site inspection during bridge construction to ensure conformance to design specifications and construction procedures.

- ☐ Other (please specify) ______________________
  ____

4. I believe that additional attention to bridge durability and longevity in the <u>maintenance phase</u> is currently needed to achieve significantly greater bridge durability/longevity.

- ☐ Yes
- ☐ No

Please check the areas below where you think additional attention would pay good dividends in bridge longevity.

- ☐ More comprehensive and/or thorough biennial inspections
- ☐ Acting more quickly on problems identified during inspections
- ☐ Greater attention to periodic routine preventive maintenance activities
- ☐ Keeping all bridge joints clean and open
- ☐ More permanent and effective repair procedures in cases of unplanned bridge damages
- ☐ Maintenances of bridge foundation scour control "system".
- ☐ Other (please specify) ______________________ ______________________

5. I believe that the type of bridge material is a significant parameter affecting bridge durability performance (for some bridge materials).

- ☐ Yes
- ☐ No

Please rate the common bridge materials below from 1-10 based on durability/longevity performance (10 = most durable; 1 = least durable). Place a numerical rating in each box.

- ☐ Reinforced Concrete
- ☐ Prestressed Concrete
- ☐ Steel
- ☐ Timber

6. I believe that the type of bridge (combination of structural form and bridge material) is a significant parameter affecting bridge durability performance.

- ☐ Yes
- ☐ No

- ☐ Intermediate Bent Caps
- ☐ Intermediate Bents/Piers/Footings
- ☐ Other (please specify) ____________________

8. I believe an important question that should be asked and addressed is the following:

____

____

____

My answer and/or guidance on this question is as follows:

____

____

____

Your name/position/address/telephone number (optional)

Name: ____________________

Position: ____________________

Address: ____________________

____

Telephone: ____________________

Thank you for taking the time to share your expertise with us!

Please return this questionnaire in the self addressed and stamped envelope provided.

Return To:

Dr. G. Ed Ramey
Department of Civil Engineering
Harbert Engineering Center
Auburn University, AL 36849

Appendix C

Listing of Bridges
in AU Subset 143

Table C-1. AU Subset 143
ALDOT Bridge Replacements
1981-1993

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
1	2-36-6.0A	1933	1985	52
	2-45-25.4A	1933	1986	53
	2-45-25.6A	1933	1986	53
	20-45-1.7A	1956	1992	36
	35-36-9.5	1930	1983	53
	53-52-8.0B	1931	1985	54
	74-22-17.9	1912	1993	81
	91-22-15.1	1935	1988	53
	20-45-1.7B	1965	1990	25
	20-45-5.5A	1956	1990	34
	I65-22-19.3	1965	1990	25
	20-45-5.5B	1965	1990	25
2	20-40-4.5	1929	1989	60
	24-40-4.6	1929	1991	62
	118-47-17.1B	1924	1982	58
	233-47-1.8	1924	1985	61
	118-47.19.1	1934	1991	57
3	4-64-25.1A	1940	1990	50
	1-59-28-7.7A	1963	1993	30
	1-59-28-7.7B	1963	1992	29
	1-59-28-7.8A	1963	1993	30
	1-59-28-7.8B	1963	1993	30
	1-59-28-7.8C	1963	1993	30
	1-59-28-7.8D	1963	1993	30
	76-59-4.0	1930	1992	29

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
3--Con't	1-65-05-3.4A	1959	1992	33
	1-65-05-3.4B	1959	1992	33
	1-65-05-6.8A	1959	1992	33
	1-65-05-6.8B	1959	1992	33
	25-59-51.1	1935	1990	55
4	1-41-20.8	1961	1989	28
	4-15-2.6	1929	1987	58
	9-14-11.7	1942	1981	39
	9-19-19.8	1924	1986	62
	21-61-22.7	1933	1987	54
	21-61-24.2	1933	1987	54
	49-62-32.1	1941	1985	44
	74-8-1.6	1931	1991	60
	74-8-2.2	1931	1991	60
	74-8-6.9	1931	1991	60
	259-62-1.1	1936	1992	56
	I85-09-9.8B	1959	1989	30
	I85-09-9.8A	1959	1989	30
5	102-29-2.0	1936	1986	50
	14-32-24.0	1930	1992	62
	14-32-24.1	1930	1992	62
	14-32-24.3	1930	1992	62
	14-32-24.7	1930	1992	62
	14-32-23.9	1930	1992	62
	14-33-0.8	1930	1992	62
	14-33-1.1	1930	1992	62
	14-33-0.3	1930	1992	62

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
5--Con't	14-33-1.2	1930	1992	62
	14-33-1.4	1930	1992	62
	14-33-0.6	1930	1992	62
	14-33-1.7	1930	1992	62
	14-53-16.9	1926	1990	64
	171-29-22.9	1932	1985	53
	171-63-11.7	1950	1987	37
	17-54-33.7	1936	1981	45
	18-29-22.9	1928	1984	56
	39-32-4.2	1928	1990	62
	60-33-1.3	1926	1993	67
	60-33-5.7	1926	1993	67
	6-11-8.4	1943	1989	46
	6-11-7.5	1943	1989	46
	6-11-7.8	1943	1989	46
	I-65-11-15.7A	1960	1990	30
	I-65-11-15.7B	1962	1989	27
	69-33-5.2	1935	1985	50
	14-32-12.0	1926	1986	60
6	51-06-24.3	1930	1989	59
	21-26-23.5	1931	1992	61
	3-51-16.9A	1927	1986	59
	51-06-22.1	1930	1989	59
	51-06.23.3	1930	1989	59
	51-06-22.4	1930	1989	59
	8-51-33.3	1928	1993	65
	81-44-5.2	1930	1993	63

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
6--Con't	81-44-05	1930	1993	63
	9-26-0.4A	1923	1992	69
	9-51-40.7B	1922	1993	71
	21-50-38.4	1930	1987	57
7	10-03-0.2	1930	1984	54
	10-03-1.3	1930	1987	57
	10-03-2.9	1930	1986	56
	103-31-5.0	1941	1990	49
	10-55-30.0	1930	1984	54
	10-55-28.1	1930	1987	57
	10-55-30.2	1930	1984	54
	12-23-3.5	1928	1985	57
	12-23-6.5	1928	1984	56
	123-31-5.3	1934	1984	50
	189-21-3.7	1930	1990	60
	27-23-17.4	1927	1985	58
	87-16-12.7	1931	1986	55
	95-34-12.1	1930	1993	63
	95-34-12.2	1930	1989	59
	95-34-17.5	1930	1984	54
	95-34-17.6	1930	1984	54
	95-34-1.7	1930	1990	60
	10-55-26.6	1930	1986	56
	95-34-17.7	1930	1984	54
8	12-13-4.8	1938	1988	50
	12-13-5.0	1938	1988	50
	21-50-35.0	1930	1987	57

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
8--Con't	21-50-38.3	1930	1987	57
	21-66-12.5	1939	1989	50
	25-46-1.2	1942	1991	49
	39-60-7.5	1950	1992	42
	41-50-13.4	1927	1989	62
	41-50-36.8	1929	1990	61
	114-12-3.2	1962	1989	27
	265-50-3.9	1955	1993	38
	10-46-12.0	1936	1984	48
	10-66-41.3	1929	1982	53
	10-66-40.8	1929	1992	63
	12-13-35.2	1930	1984	54
	12-13-35.3	1930	1984	54
	12-50-3.0	1931	1983	52
	12-50-3.3	1931	1985	54
	12-50-2.5	1930	1983	53
	12-50-2.7	1931	1983	52
	12-50-2.9	1931	1983	52
	178-13-1.8	1930	1987	57
9	3-27-13.4	1927	1985	58
	13-49-7.3A	1958	1986	28
	13-49-7.3B	1958	1986	28
	15-27-16.3	1929	1988	59
	17-49-1.5	1924	1991	67
	17-49-6.7	1931	1992	61
	17-49-9.5	1931	1992	61
	17-49-9.9	1931	1992	61

Division	Bridge Structure No.	Year Built	Year Replaced	Age at Replacement
9--Con't	17-49-14.6	1931	1991	61
	17-49-21.8	1931	1991	61
	17-49-23.0	1931	1991	61
	59-02-65.5	1926	1989	63
	188-49-17.0	1946	1983	37
	42-49-0.00	1932	1986	54
	16-49-33.0	1929	1991	62
	17-49-16.5	1931	1992	61
	17-49-23.7	1931	1992	61
	17-49.24.2	1931	1993	62